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HANDBOOK
REMEDIAL ACTION AT WASTE DISPOSAL SITES (REVISED)

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ABSTRACT

This handbook is intended for use as a basic reference tool on remedial action. It will assist governmental and industrial officials and technical persons in understanding remedial technologies; selecting potentially applicable technologies for a given waste site, and planning for remedial action.

The handbook begins with an overview of the remedial investigation/feasibility study (RI/FS) process as outlined in the National Contingency Plan (NCP).

The remaining sections of the document describe remedial technologies. The technologies are organized according to the type of site problem they are intended to remedy (e.g., surface water controls, groundwater controls). Emphasis is placed on those technologies which have been demonstrated for hazardous waste sites. However, less detailed information is also included on emerging technologies. Established technologies are generally described in terms of the following factors:

- General description
- Applications and limitations
- Design considerations
- Construction/implementation considerations
- Operation, maintenance, and monitoring
- Technology selection/evaluation
- Costs.

The section on applications and limitations describes waste and site conditions which favor or limit use of each technology.

Major design, construction, and operational considerations are described in subsequent sections. The information is intended to provide a basic understanding of the technologies, and a framework for planning and developing remedial action alternatives. More detailed guidance manuals and design manuals on these subjects are referenced extensively throughout the document.

The section on technology selection/evaluation provides an overview of each technology or related group of technologies in terms of their known technical feasibility (e.g., reliability, effectiveness, ease of implementation) and any significant environmental, public health, or institutional effects associated with them.

Finally, unit cost data, expressed mainly in 1984 or 1985 dollars, are included for each technology. Cost data from hypothetical sites or actual case histories has also been included where it is appropriate.

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SECTION 1.0

INTRODUCTION

1.1 Background and Objectives

The Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA) provides the U.S. Environmental Protection Agency (EPA) with the authority and responsibility to establish procedures for evaluating response actions, determining their appropriateness for specific uncontrolled waste sites, and implementing cost-effective responses. The plan for enacting these provisions appears in the (proposed) revised National Contingency Plan (NCP) issued in the Federal Register on February 12, 1985. In order to carry out this mandate, the USEPA, Office of Research and Development (USEPA/ORD) is involved in extensive research, development, and documentation of existing and emerging remedial technologies.

The first guidance document to be published by this office on remedial technologies was the Handbook of Remedial Action at Waste Disposal Sites, published in 1982. This document has been recognized nationwide as a central reference tool for understanding and planning remedial actions. Since 1980, when research for the Handbook was actually completed, there have been numerous advances in the state-of-the-art of remedial actions. Remedial actions have been initiated at hundreds of federally- and industrially-financed cleanups and these actions are providing considerable insight into their design, reliability, effectiveness, and cost. In addition, the EPA and private industry are sponsoring a great deal of research to advance the state-of-the-art of remedial actions. Further, the EPA has prepared numerous detailed guidance documents which can be used in the planning and preliminary design of specific remedial action technologies.

Not only have there been important technology advances over the past five years, but there have also been significant developments in formalizing the remedial action selection process. The NCP requires that a detailed remedial investigation (RI) and feasibility study (FS) be carried out for all federally-funded remedial actions in order to ensure selection of cost-effective remedial alternatives. The EPA has developed detailed guidance on carrying out the various steps of the RI/FS process.

In light of these recent advances in the state-of-the-art of remedial responses, it became necessary to revise and update the Handbook so that it may continue to be used effectively as a central reference on remedial technologies. This document is the revised version of the original Handbook and provides current data on remedial technologies and their costs.

The Handbook is intended to familiarize federal, state, and local officials and private industry with the range of remedial technologies; their applications and limitations; major design and construction considerations; and approximate costs. It provides sufficient information on remedial technologies to enable the user to select potentially applicable technologies for a given site, and understand what is involved in designing and implementing these technologies. It also enables the user to make an easy transition to more detailed guidance documents and design manuals; reference is made to these documents throughout the Handbook.

1.2 Organization of the Handbook

The Handbook is organized in two parts: Section 2 provides an overview of the RI/FS process, and Sections 3 through 11 provide specific information on the remedial technologies.

The remedial technologies included in the Handbook are grouped into one of nine categories largely depending upon the type of site problem they are intended to remedy. These categories include:

- Surface Water Controls (Section 3)
- Air Pollution Controls (Section 4)
- Groundwater Controls (Section 5)
- Gas Migration Controls (Section 6)
- On-Site and Off-Site Disposal of Wastes and Soil (7)
- Contaminated Sediment (Section 8)
- In-Situ Treatment Measures (Section 9)
- Direct Waste Treatment (Section 10)
- Contaminated Water and Water and Sewer Liners (Section 11)

Emphasis is placed on those technologies which are demonstrated for hazardous waste site applications. Many emerging technologies are also included but in less detail.

The discussion of each established technology or group of similar technologies is organized as follows:

- General description
- Applications/limitations
- Design considerations
- Construction/implementation considerations
- Operation, maintenance and monitoring

- Technology selection/evaluation
- Unit costs

The general description provides a basic overview of the technology and its intended use. Applications and limitations describe mainly those waste- and site-specific characteristics that favor, preclude, or limit use of the technology. For example, presence of high permeability soils favors in-situ treatment whereas low permeability soils limit use of these technologies. This section also addresses the status of the technology (e.g., well developed, emerging). The information in this section is particularly useful in screening out those technologies that are not applicable for a specific site.

The sections on design; construction and implementation; and operation, maintenance, and monitoring are intended to provide a basic understanding of the technology, the equipment and materials involved, and the major considerations in implementing, operating and maintaining the system. While these sections provide a framework for planning and developing remedial action alternatives, they are not intended for use in detailed design. The information should not be substituted for the detailed guidance and design manuals referenced throughout the document nor for the services of a qualified design engineer. Table 1-1 lists the major guidance documents which complement the data presented in this handbook.

The section on technology selection/evaluation provides a summary of the technical suitability of the technology based on established performance, reliability, implementability, and safety. It also makes mention of any environmental, public health, or institutional concerns which may be particularly noteworthy.

The final section on each technology or group of technologies presents unit cost data and costs for actual or hypothetic remedial actions. The unit cost data has been expressed in 1984 or 1985 dollars.

TABLE 1-1.

GUIDANCE DOCUMENTS AND MANUALS RELATED TO REMEDIAL ACTIONS AT UNCONTROLLED WASTE SITES

Leachate Plume Management, in press. (USEPA, 1985)

Slurry Trench Construction for Pollution Migration Control, EPA-540/2-84-001. (Spooner, P. et al., 1984)

Covers for Uncontrolled Hazardous Waste Sites, EPA-540/2-85/002. (U.S. Army Engineer Waterways Experiment Station, 1985)

Evaluating Cover Systems for Solid and Hazardous Waste, SW-867C, NTIS-PB81-166340 (Lutton, R.J., 1982)

Guide to Disposal of Chemically Stabilized and Solidified Waste, SW-872, NTIS-PB81-181505. (U.S. Army Engineer Waterways Experiment Station, 1982)

Evaluation of Systems to Accelerate the Stabilization of Waste Piles and Deposits, in press. (USEPA, 1985)

Review of In-Place Treatment Techniques for Contaminated Surface Soils, EPA-540/2-84-003a. (USEPA, 1984)

Compatibility of Grouts with Hazardous Waste, EPA 600/2-84-015. (Spooner, P. et al., 1984.)

Case Studies - Remedial Response at Hazardous Waste Sites, EPA-540/2-84-002a and 002b. (USEPA, 1984)

Drum Handling Practices at Hazardous Waste Sites, in press. (USEPA, 1985)

Fugitive Dust Control at Hazardous Waste Sites, in press. (USEPA, 1985)

Guidance Manual on Overtopping Controls for Hazardous Waste Impoundments, in press. (USEPA, 1985)

Guidance on Remedial Investigations Under CERCLA, EPA-540/G-85-002 (USEPA, 1985)

Guidance on Feasibility Studies Under CERCLA, EPA-540/G-85/003 (USEPA, 1985)

Modeling Remedial Responses at Uncontrolled Hazardous Waste Sites, EPA-540/2-85-001. (USEPA, 1985)

Technical Handbook for Stabilization/Solidification of Hazardous Waste, in production. (U.S. Army Engineer Waterways Experiment Station, 1985)

Hazardous Waste Processing Technology, Ann Arbor Science Publishers, Inc., Ann Arbor, MI

The Remedial Action Costing Procedures Manual, in press. (USEPA, 1985).

SECTION 2

REMEDIAL ACTION SELECTION PROCESS

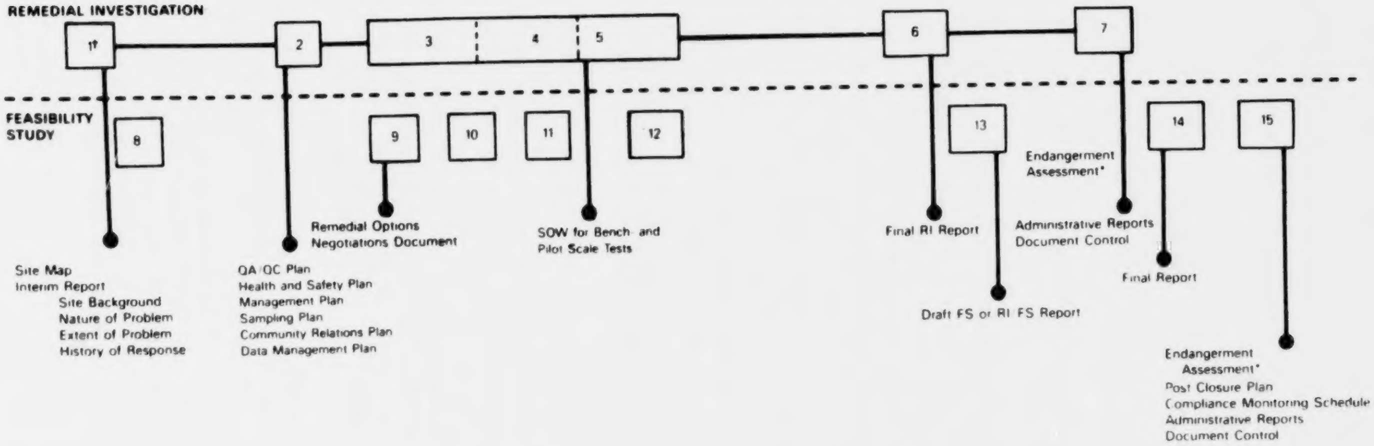
2.1 Remedial Investigation/Feasibility Study Process

Section 105 of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) requires that the EPA establish procedures to ensure that the Hazardous Substance Response Trust Fund (commonly known as Superfund) be used as effectively as possible in responding to releases of hazardous substances in the environment. In accordance with Section 105 of CERCLA, the EPA has established a process for discovering releases, evaluating remedies, determining the appropriate extent of response, and ensuring that remedies selected are cost-effective. This process is commonly referred to as the remedial investigation/feasibility study (RI/FS) process, and is outlined in Section F of the revised National Contingency Plan (NCP). The EPA has developed detailed guidance on the RI/FS process in the form of two separate guidance documents: Guidance on Remedial Investigations under CERCLA (USEPA, 1985a), and Guidance on Feasibility Studies under CERCLA (USEPA, 1985b).

For every site that is targeted for remedial response action under Section 104 of CERCLA, the NCP requires that a detailed RI/FS be conducted. The RI emphasizes data collection and site characterization. Its purpose is to define the nature and extent of contamination at a site to the extent necessary to evaluate, select, and design a cost-effective remedial action. The FS emphasizes data analysis and decision making; it uses the data from the RI to develop response objectives and alternative remedial responses. These alternatives are then evaluated in terms of their engineering feasibility, public health protection, environmental impacts, and costs.

The remedial investigation and feasibility study are interdependent processes and are generally performed concurrently rather than sequentially. Figure 2-1 presents a flow chart of the RI/FS process, illustrating the major steps and the interdependence and concurrence of tasks in the RI and the FS. The numbers identifying the tasks (boxes) in the flow chart are keyed to the tasks in the model statements of work for the RI and FS, and are tabulated under the flow chart. The interactive nature of the RI/FS process is due to the integration of data collected during the RI and the data analysis activities performed during the FS.

FIGURE 2-1.
RI/FS PROCESS



Remedial Investigation

Model Statement of Work
for Remedial Investigations

- Task #1 Description of Current Situation
- Task #2 Plans & Management
- Task #3 Site Investigation
- Task #4 Site Investigation Analysis
- Task #5 Laboratory & Bench Scale Studies
- Task #6 Reports
- Task #7 Community Relations Support

Feasibility Study

Model Statement of Work
for Feasibility Studies

- Task #8 Description of Proposed Response
- Task #9 Preliminary Remedial Technologies
- Task #10 Development of Alternatives
- Task #11 Initial Screening of Alternatives
- Task #12 Evaluation of Alternatives
- Task #13 Preliminary Report
- Task #14 Final Report
- Task #15 Additional Requirements

---

† Numbers in the boxes refer to tasks described in the Model Statement of Work for RI/FS under CERCLA Guidance issued February 1985

* Endangerment assessments may be prepared at any point in the RI/FS Process in support of enforcement actions

2.2 Remedial Action Selection Process (Feasibility Study)

Since a primary purpose of this Handbook is to assist the user in conducting the feasibility study, this section discusses the FS process steps outlined in Figure 2-1 in more detail.

2.2.1 Proposed Response Action Identification

The first step in the FS is to identify existing site problems using preliminary RI data (site background data, previous studies, initial RI investigation activities) and to determine the categories of remedial technologies (e.g., surface water controls, air pollution controls) that are most applicable. Figure 2-2 presents a matrix of remedial technology categories matched with specific site problems. This categorization of remedial technologies corresponds to the categorization found in Sections 3 through 11 of this Handbook. As illustrated in the matrix, more than one technology category may be applicable at a given site.

2.2.2 Technology Screening

The next step in the FS process is to identify and screen potentially applicable remedial technologies from the general remedial technology categories selected. Technologies are screened to eliminate those that may prove difficult to implement, rely on unproven technologies, or may not achieve the remedial objectives (established early in the RI/FS process) within a reasonable time period. This screening process focuses on eliminating those technologies which have severe limitations for a given set of waste- and site-specific conditions. The screening step may also eliminate technologies based on inherent technology limitations. Site, waste, and technology characteristics which are used to screen inapplicable remedial technologies are described in more detail below:

- Site Characteristics. Site data should be reviewed to identify conditions that may limit or promote the use of certain remedial technologies. Such information is generally gathered during the RI. Figure 2-3 provides a comprehensive list of site characterization data which is necessary for screening and detailed evaluation of remedial technologies. Technologies whose use is clearly precluded by site characteristics should be eliminated from further consideration. For example, the presence of very low permeability soils would generally preclude the use of in-situ methods since it would be impossible to ensure complete mixing of treatment reagents with waste components. Similarly, the presence of contaminated sediments at depths below 65 feet would preclude the use of certain types of dredges.

FIGURE 2-2
MATRIX OF GENERAL REMEDIAL TECHNOLOGY CATEGORIES FOR SPECIFIC SITE PROBLEMS

	Remedial Technology Categories								
Site Problem	Surface Water Controls (See Section 3)	Air Pollution Controls (See Section 4)	Leachate and Groundwater Controls (See Section 5)	Gas Migration Control (See Section 6)	Waste and Soil Excavation and Removal and Land Disposal (See Section 7)	Contaminated Sediments Removal and Containment (See Section 8)	In-Situ Treatment (See Section 9)	Direct Waste Treatment (See Section 10)	Contaminated Water Supply and Sewer Line Controls (See Section 11)
Volatilization of chemicals into air		●							
Hazardous particulates released to atmosphere		●							
Dust generation by heavy construction or other site activities		●							
Contaminated site run-off	●								
Erosion of surface due to wind or water	●								
Surface seepage of leachate	●								
Flood hazard or contact of surface water body with wastes	●								
Leachate migrating vertically or horizontally			●				●		
High water table which may result in groundwater contamination or interfere with other remedial technologies			●						
Precipitation infiltrating into site to form leachate	●		●						
Evidence of methane or toxic gases migrating laterally underground				●					
On-site waste materials in non-disposed form: drums, lagooned waste, wastepiles					●		●	●	
Contaminated surface water, groundwater or other aqueous or liquid waste					●		●	●	
Contaminated soils					●		●	●	
Toxic and/or hazardous gases which have been collected								●	
Contaminated stream banks and sediments					●	●		●	
Drinking water distribution system contamination									●
Contaminated sewer lines									●

FIGURE 2-3.
IDENTIFICATION OF DATA NEEDS FOR VARIOUS REMEDIAL TECHNOLOGIES

	AIR POLLUTION CONTROLS		SURFACE WATER CONTROLS				LEACHATE AND GROUND-WATER CONTROLS			
REQUIRED DATA ITEMS	Capping	Dust Control Measures	Capping	Grading	Revegetation	Diversion and Collection Systems	Capping	Containment Barriers	Groundwater Pumping	Subsurface Collection Drains
GENERAL SITE CONDITIONS										
Accessibility	•	•	•	•	•	•	•	•	•	•
Topography	•	•	•	•	•	•	•	•		
Native Vegetation	•	•	•	•	•	•	•	•		
WASTE CHARACTERISTICS										
Physical State	•	•	•	•	•	•	•	•	•	•
Chemical Composition	•	•	•	•	•	•	•	•	•	•
Disposal/Burial Practices	•	•	•	•	•	•	•	•	•	•
Physical/Chemical Properties	•	•	•		•	•	•	•	•	•
SITE GEOLOGY										
Seismic History	•		•	•		•	•	•		•
Depth to Bedrock	•		•	•			•	•	•	•
Bedrock Type			•	•				•	•	•
Bedrock Profile								•	•	•
Structural Configuration								•	•	•
Structural Strength	•		•	•		•	•	•		
Bedrock Permeability/Porosity								•	•	•
SOIL CHARACTERISTICS										
Profiles to Bedrock	•		•	•	•	•	•	•	•	•
Type/Texture	•	•	•	•	•	•	•	•		•
Permeability/Porosity	•	•	•	•	•	•	•	•	•	•
Engineering Properties	•		•	•		•	•	•		
Soil Chemistry	•	•	•		•		•	•		
Erosion Potential	•	•	•	•	•	•	•	•		
Contaminant Profiles	•	•	•	•	•	•	•	•	•	•
Moisture Content	•	•	•		•	•	•	•		•
GROUNDWATER CHARACTERISTICS										
Natural Groundwater Chemistry								•	•	•
Seasonal Potentiometric Surfaces	•	•	•	•	•	•	•	•	•	•
Aquifer Profile								•	•	•
Aquifer Characteristics								•	•	•
Groundwater Velocity and Direction of Flow								•	•	•
Groundwater Recharge and Discharge Areas							•	•	•	•
Contaminant Profiles								•	•	•
Supply Well Characteristics								•	•	•
SURFACE WATER										
Proximity of Nearest Surface Waters	•		•	•		•	•	•	•	•
Presence of Leachate Seeps	•		•	•	•	•	•	•	•	•
Floodplain or Coastal Storm Surge Boundaries	•		•	•	•	•	•	•	•	•
Stream Profiles	•			•		•				
Surface Water Use	•			•		•		•	•	•
Drainage Area/Runoff				•		•				
Local Surface Water Quality				•		•				
Stream Flow Characteristics				•		•		•	•	•
CLIMATOLOGY										
Evapotranspiration Parameters	•	•			•	•	•			
Wind Speed and Direction	•	•			•		•			
Temperature Parameters	•	•			•					
Precipitation	•	•	•	•	•	•	•	•	•	•
Local Air Quality	•	•								
Regional Air Quality										

FIGURE 2-3. (CONTINUED)

	GAS MIGRATION CONTROLS			WASTE EXCAVATION AND REMOVAL	CONTAMINATED SEDIMENT REMOVAL AND CONTAINMENT		IN-SITU TREATMENT
REQUIRED DATA ITEMS	Capping	Collection (with or without Recovery)	Containment	Soil and Waste Excavation and Removal	Removal	Containment and Turbidity Control	In-Situ Treatment
GENERAL SITE CONDITIONS							
Accessibility	•		•	•	•	•	•
Topography	•	•	•	•			•
Native Vegetation	•			•			•
WASTE CHARACTERISTICS							
Physical State	•	•	•	•	•	•	•
Chemical Composition	•	•	•	•	•	•	•
Disposal/Burial Practices	•	•	•	•	•	•	•
Physical/Chemical Properties	•	•	•		•	•	•
SITE GEOLOGY							
Seismic History	•		•				
Depth to Bedrock	•		•	•			•
Bedrock Type							•
Bedrock Profile							•
Structural Configuration							•
Structural Strength	•		•				
Bedrock Permeability/ Porosity							•
SOIL CHARACTERISTICS							
Profiles to Bedrock	•	•	•	•		•	•
Type/Texture	•	•	•	•	•	•	•
Permeability/Porosity	•	•	•	•	•	•	•
Engineering Properties	•			•	•		•
Soil Chemistry	•	•	•	•	•	•	•
Erosion Potential	•			•		•	
Contaminant Profiles	•	•	•	•	•	•	•
Moisture Content	•	•	•	•			•
GROUNDWATER CHARACTERISTICS							
Natural Groundwater Chemistry							•
Seasonal Potentiometric Surfaces	•	•	•	•			•
Aquifer Profile							•
Aquifer Characteristics							•
Groundwater Velocity and Direction of Flow							•
Groundwater Recharge and Discharge Areas				•			•
Contaminant Profiles				•			•
Supply Well Characteristics							•
SURFACE WATER							
Proximity of Nearest Surface Waters				•			•
Presence of Leachate Seeps	•	•	•	•	•	•	•
Floodplain or Coastal Storm Surge Boundaries	•	•	•	•	•	•	•
Stream Profiles					•	•	
Surface Water Use					•	•	•
Drainage Area/Runoff	•	•	•	•	•	•	•
Local Surface Water Quality					•	•	•
Stream Flow Characteristics					•	•	
CLIMATOLOGY							
Evapotranspiration Parameters	•	•	•				•
Wind Speed and Direction	•	•	•	•			•
Temperature Parameters	•	•	•	•	•	•	•
Precipitation	•	•	•	•			•
Local Air Quality		•		•	•		•
Regional Air Quality							

- Waste Characteristics. Identification of waste characteristics that limit the effectiveness or feasibility of the remedial technologies is an important part of the screening process. Table 2-1 presents waste characteristics that may influence the feasibility and effectiveness of remedial technologies. Technologies clearly limited by these waste characteristics should be eliminated from consideration. Waste characteristics particularly affect the feasibility of in-situ methods, direct treatment methods, and land disposal (on- or off-site).

- Technology Limitations. During the screening process, the level of technology development, performance record, and inherent construction, operation, and maintenance problems should be identified for each technology considered. Technologies that are unreliable, perform poorly, or are not fully demonstrated may be eliminated in the screening process. For example, certain grouting methods and in-situ methods have not been developed to a point where they can be implemented in the field without extensive research.

TABLE 2-1
WASTE CHARACTERISTICS THAT MAY AFFECT
REMEDIAL TECHNOLOGY SELECTION

Quantity/concentration
Chemical composition
Acute toxicity
Persistence
Biodegradability
Radioactivity
Ignitability
Reactivity/corrosivity
Infectiousness
Solubility
Volatility
Density
Partition coefficient
Compatibility with other chemicals
Treatability

For each remedial technology or group of technologies described in Sections 3 through 11 of this Handbook, a discussion of applications and limitations has been included to assist the user in screening inappropriate technologies.

During the screening process, the user should rely on a comprehensive list of technologies classified by their remedial category to assure that all viable technologies are being considered. This list should be periodically updated to incorporate newly developed or developing technologies.

2.2.3 Development of Remedial Action Alternatives

Technologies which have passed the initial screening process can then be combined to form overall remedial action alternatives. These alternatives represent a workable number of options that each appear to adequately address all site problems. Each alternative may consist of an individual technology or a combination of technologies. An example of an alternative for a groundwater contamination problem for which removal and leachate/groundwater controls are applicable may include the following: partial removal of contaminated soil ("hot spots")/disposal of soil in a permitted landfill/360° slurry wall/groundwater pumping system/leachate treatment using activated carbon/cap.

Additional guidance on developing remedial action alternatives and specific formulative criteria that must be met can be found in Guidance on Feasibility Studies Under CERCLA (USEPA, 1985b).

2.2.4 Screening and Detailed Evaluation of Remedial Action Alternatives

Once a list of remedial action alternatives has been developed to deal with all site problems, these alternatives undergo screening to eliminate those which are an order of magnitude more costly than other alternatives and/or provide inadequate public health protection or have adverse environmental impacts which preclude their use. Only alternatives that satisfy the remedial response objectives and contribute substantially to the protection of public health, welfare, and the environment should be considered further.

Those alternatives remaining after the screening are then subject to detailed evaluation. The objective of this detailed analysis is to provide a level of evaluation to support the selection of the most cost-effective alternative as required by the NCP. The NCP specifies that the EPA select the lowest cost alternative which effectively mitigates and minimizes damages and provides adequate protection of public health, welfare, and the environment. In order to meet this goal, each alternative is analyzed in terms of the following factors:

- Technical considerations
- Environmental concerns
- Public health concerns
- Institutional concerns
- Costs.

2.2.4.1 Technical Considerations

Technical suitability of an alternative is evaluated in terms of performance, reliability, implementability, time, and safety.

a. Performance

Two aspects of remedial actions determine their desirability on the basis of performance: effectiveness and useful life.

Effectiveness is evaluated in terms of the ability to perform intended functions, such as containment, diversion, removal, destruction, or treatment. The effectiveness of alternatives should be determined either through design specifications or by performance evaluation.

It should be established which environmental and public health standards and criteria are applicable at the site and the proposed alternatives should be evaluated according to those standards. Any specific waste or site characteristics which could potentially impede effectiveness should be considered. The evaluation should also consider the effectiveness of combinations of technologies.

Useful life is defined as the length of time the level of effectiveness can be maintained. Most remedial technologies, with the exception of destruction, deteriorate with time. Often, deterioration can be slowed through proper system operation and maintenance, but the technology eventually may require replacement. Each alternative should be evaluated in terms of the projected service lives of its component technologies. Resource availability in the future life of the technology, as well as appropriateness of the technologies, must be considered in estimating the useful life of the project.

b. Reliability

Two aspects of remedial technologies that provide information about reliability are their operation and maintenance requirements and their demonstrated reliability at similar sites.

Operation and maintenance requirements include the frequency and complexity of necessary operation and maintenance. Technologies requiring frequent or complex operation and maintenance activities should be regarded as less reliable than technologies requiring little or straightforward operation and maintenance. The availability of labor and materials to meet these requirements should also be considered.

Demonstrated and expected reliability is a way of measuring the risk and effect of failure. The engineer should evaluate whether the technologies have been used effectively at similar sites; whether the combination of technologies have been used together effectively; whether failure of any one technology has an immediate impact on receptors; and whether the alternative has the flexibility to deal with uncontrollable changes at the site.

c. Implementability

Another important aspect of remedial alternatives is implementability, which can be described as the relative ease of installation (constructability) and the time required to achieve a given level of response.

Constructability is determined by conditions both internal and external to the site conditions and include such items as location of underground utilities, depth to water table, heterogeneity of subsurface materials, and location of the site (i.e., remote location vs. a congested, urban area). The engineer should evaluate what measures can be taken to facilitate construction under these conditions. External factors which affect implementation include the need for special permits or agreements, equipment availability, and the location of suitable off-site treatment or disposal facilities.

Time has two components that should be addressed: (1) the time it takes to implement a remedy, and (2) the time it takes to actually see beneficial results. Beneficial results are defined as the reduction of contaminants to some acceptable, pre-established level.

d. Safety

Each remedial alternative should be evaluated with regard to safety. This evaluation should include threats to the safety of nearby communities and environments as well as those to workers during implementation. Factors to consider are fire, explosion, and exposure to hazardous substances.

2.2.4.2 Institutional Concerns

The effects of Federal, State, and local standards and other institutional considerations on the implementation and operational timing of each alternative should be determined. In general, it is expected that regulatory programs under the Resource Conservation and Recovery Act (RCRA), the Safe Drinking Water Act (SDWA), the Federal Water Pollution Control Act (Clean Water Act or CWA), and the Toxic Substances Control Act (TSCA) will have the broadest applications to remedial action alternatives.

2.2.4.3 Public Health Concerns

All remedial sites should also undergo a public health evaluation, although the form and extent of the assessment will depend on the situation. Steps which may be included in the public health evaluation are:

- Baseline site evaluation - Preliminary evaluation and classification that all sites must undergo, i.e., current site conditions.
- Exposure assessment - Analysis of the extent and duration of human exposure to site contaminants in the absence of remedial action.
- Standards analysis - Comparison of projected environmental concentrations to appropriate ambient standards or criteria.
- Evaluation of alternatives - Analysis and comparison of each alternative in terms of its ability to remove or mitigate exposures of concern.

2.2.4.4 Environmental Concerns

The environmental assessment of the proposed alternatives is intended to determine any adverse impacts of these alternatives, methods for mitigating these impacts, and cost of mitigation. The environmental assessment will generally consider the following for each alternative: (1) effects on environmentally sensitive areas; (2) exceedance of environmental standards; (3) short- and long-term effects, both beneficial and adverse; (4) irreversible commitments of resources; and (5) mitigative measures and their costs. The environmental assessment should also analyze the environmental impacts in the absence of remedial investigation.

2.2.4.5 Costs

The detailed analysis of costs should include estimated capital and operation and maintenance costs. It should also include a present worth analysis and a sensitivity analysis which assesses the effects that variations in specific assumptions associated with design, construction, and operation can have on the estimated costs of an alternative. The Remedial Action Costing Procedures Manual (USEPA, 1985c) provides detailed procedures for developing costs.

Direct capital costs include equipment, labor and materials necessary for installation or construction of remedial actions. This includes costs for the following (USEPA, 1985C):

- Construction Costs
  - Equipment
  - Labor
  - Materials
- Equipment Costs
  - Installed
  - Purchased
- Land and Site Development
  - Equipment
  - Labor
  - Materials
- Building and Services
  - Equipment
  - Labor
  - Materials
- Population Relocation Costs.

Indirect Capital Costs consist of engineering costs and contingency allowances.

Engineering expenses, include administration and supervision, design and development, monitoring and testing, and project and cost engineering. Typically, engineering expenses are in the range of 7 to 15 percent of total direct capital costs. However, certain sites will require additional resources for legal fees, obtaining licenses and permits, and start-up and shake-down of equipment (USEPA, 1985C).

Contingency allowances are added to total capital costs to account for unforseen circumstances such as adverse weather conditions, and inadequate site characterization which would result in increased captial costs. Contingency allowances typically range from 15 to 25 percent of total capital costs (USEPA, 1985C).

Operation and maintenance (O&M) costs are those post-construction/ installation costs necessary to ensure continued effectiveness of a remedial action. The following cost components are included (USEPA, 1985C):

- Operating Labor - wages, salaries, training, overhead, and fringe benefits associated with the labor needed for post-construction operations.

- Maintenance Materials and Labor - labor, parts, and other materials required to perform routine maintenance of facilities and equipment.

- Auxiliary Materials and Energy - such items as chemicals and electricity needed for plant operations, water and sewer service, and fuel costs.

- Purchased Services - such items as sampling costs, laboratory fees, and other professional services.

- Disposal - transportation and disposal of any waste materials, such as treatment plant residues, generated during the course of a remedial action.

- Administrative costs - administration of remedial action operation and maintenance not included under other categories such as labor overhead.

- Insurance, Taxes and License - liability and sudden and accidental insurance; real estate taxes on purchased land or right-of-way (for private-lead actions); licensing fees for certain technologies; and permit renewal and reporting costs.

- Maintenance reserve and contingency costs - annual payments into escrow funds to cover anticipated replacement or rebuilding of equipment and any large unanticipated O&M costs, respectively (for private lease actions).

- Other Costs - Includes all other items which do not fit into any of the above categories.

2.2.5 Remedial Alternative Selection

Each of the alternatives is summarized relative to each of the criteria described in Section 2.2.4. This summary is generally in the form of tables which allow the alternatives to be compared easily so decisionmakers can select the most appropriate alternative.

REFERENCES

USEPA. 1985a. Guidance on Remedial Investigations under CERCLA. Hazardous Waste Engineering Research Laboratory, Cincinnati, OH and Office of Emergency and Remedial Response, Washington, DC.

USEPA. 1985b. Guidance on Feasibility Studies Under CERCLA. Hazardous Waste Engineering Research Laboratory, Cincinnati, OH and Office of Emergency and Remedial Response, Washington, DC.

USEPA. 1985c. The Remedial Action Costing Procedures Manual. Hazardous Waste Engineering Research Laboratory, Cincinnati, OH and Office of Emergency and Remedial Response, Washington, DC.

SECTION 3

SURFACE WATER CONTROLS

Surface water controls include a wide range of containment, diversion, and collection methods which are designed to minimize contamination of surface waters, prevent surface water infiltration, and prevent off-site transport of surface waters which have been contaminated.

Surface water control technologies are designed to perform one of six basic functions (Ehrenfelder and Bass, 1983).

- Prevention of Run-on/Interception of Run-off: These technologies are used to divert or intercept surface water. Technologies which are designed to prevent or reduce run-on include: dikes, diversion channels, floodwalls, terraces, grading, and revegetation. Temporary diversion dikes, diversion channels, and terraces are diversion measures constructed upslope of a site to direct run-on from off site to a collection system or away from the site. Terraces are used in combination with dikes or ditches to channel water stopped by the terraces away from the site.

- Prevention of Infiltration: The primary method for preventing infiltration of on-site surface water is capping. Grading also helps to minimize infiltration by maximizing the amount of water which will run off without causing significant erosion. Revegetation can either promote or minimize infiltration.

- Control of Erosion: Surface water control technologies reduce erosion by reducing slope length, slope steepness, or improving soil management. Innterception dikes, diversion channels, and terraces can be used to reduce slope length. Slope steepness is controlled by proper grading.

- Collection and Transfer of Water: Collection and transfer technologies serve to collect water which has been diverted away from the site or prevented from infiltrating, and discharge or transfer the collected water to storage or treatment. Chutes (or flumes) and downpipes are designed to transfer water away from diversion structures such as dikes or terraces to stabilized channels or outlets. Waterways can be used to intercept or divert water as well as collect and transfer water diverted elsewhere. These waterways form the base of the surface water collection system. They collect water from diversion structures, chutes, downpipes and ditches and channel it either to treatment or discharge.

- Storage and Discharge of Water: Water storage and discharge technologies include seepage basins and ditches, sedimentation basins, and storage ponds. Their function will depend on the level of contamination of the water they receive. Seepage basins and ditches are used to discharge uncontaminated or treated water downgradient of the site. Sedimentation basins are used to control suspended solid particles in surface water flow.

- Protection from Flooding: Flood control dikes (or embankments) and floodwalls are flood protection measures constructed as perimeter structures surrounding a waste site to isolate the site from floodwaters. Embankments in areas subject to river flooding are called levees. They are more expensive than run-off dikes or terraces and will usually be used only in areas where flooding is likely to be a problem. Floodwalls, which are more expensive than levees, can be used at sites with insufficient land area to construct a levee. Lagoon covers are synthetic liners which are placed atop of impoundment to prevent overtopping.

The most effective strategy for managing surface flow frequently includes a combination of several of these water control technologies. Table 3-1 summarizes the major surface water controls discussed in this section and indicates their primary function.

3.1 Capping

3.1.1 General Description

Capping is a process used to cover buried waste materials to prevent their contact with the land surface and groundwater. The designs of modern caps usually conform to the performance standards in 40 CFR 264.310, which addresses RCRA landfill closure requirements. These standards include minimum liquid migration through the wastes, low cover maintenance requirements, efficient site drainage, high resistance to damage by settling or subsidence, and a permeability lower than or equal to the underlying liner system or natural soils. These performance standards may not always be appropriate, particularly in instances where the cap is intended to be temporary, where there is very low precipitation, and when the capped waste is not leached by infiltrating rainwater.

There are a variety of cap designs and capping materials available. Most cap designs are multi-layered to conform with the above-mentioned design standards, however, single-layered designs are also used for special purposes. The selection of capping materials and a cap design is influenced by specific factors such as local availability and costs of cover materials, desired functions of cover materials, the nature of the wastes being covered, local climate and hydrogeology, and projected future use of the site in question.

TABLE 3-1.
SUMMARY OF SURFACE WATER CONTROLS

	Primary Function					
Technology	Prevent or Intercept Run-on/Run-off	Prevent or Minimize Infiltration	Reduce Erosion	Collect and Transfer Water	Protection from Flooding	Discharge Water
Capping		X				
Lagoon Covers		X				
Grading	X		X			
Revegetation	X	X	X			
Dikes and Berms	X		X		X	
Channels and Waterways			X	X		
Terraces and Benches	X		X			
Chutes and Downpipes			X	X		
Seepage Basins and Ditches						X
Sedimentation Basins and Ponds	X					X
Levees and Floodwalls	X				X	

3.1.2 Applications/Limitations

Capping is necessary whenever contaminated materials are to be buried or left in place at a site. In general, capping is performed when extensive subsurface contamination at a site precludes excavation and removal of wastes because of potential hazards and/or unrealistic costs.

Capping is often performed together with the groundwater extraction or containment technologies described in Section 5 to prevent, or significantly reduce further plume development; thus reducing the time needed to complete groundwater cleanup operations. In addition, groundwater monitoring wells are often used in conjunction with caps to detect any unexpected migration of the capped wastes. A gas collection system should always be incorporated into a cap when wastes may generate gases (see Section 6). Capping is also associated with surface water control technologies such as ditches, dikes and berms because these structures are often designed to accept rainwater drainage from the cap. Two other surface water control technologies, grading and revegetation, are incorporated into multi-layered caps.

The main disadvantages of capping are the need for long-term maintenance and uncertain design life. Any caps will need to be periodically inspected for settlement, ponding of liquids, erosion, and naturally occurring invasion by deep-rooted vegetation. In addition, the groundwater monitoring wells, often associated with caps, need to be periodically sampled and maintained. However, these long-term maintenance requirements usually are considerably more economical than excavation and removal of the wastes.

The design life of a cap is uncertain because of the uncertain life of synthetic liner materials (if one is used in the cap), the uncertain amounts of annual rainfall which will infiltrate natural and admixed liner materials, and the uncertain rate of waste migration which would result from any infiltrating rainwater. This uncertainty may necessitate the strategic placement of monitoring wells at a site to detect any waste migration, thus signaling the need to replace the cap. Caps generally have a minimum design life of 20 years when a synthetic liner is the only liquid barrier. This period may extend to over one hundred years when a synthetic liner is supported by a low-permeability base; the underlying wastes are unsaturated; there is great distance between the waste and the groundwater table; and proper maintenance procedures are observed. Rigid barriers such as concrete and bituminous membranes are vulnerable to cracking and chemical deterioration, but the cracks can be exposed, cleaned, and repaired (sealed with tar) with relative ease. Concrete covers may have a design life of about 50 years, except when applied to chemically severe or physically unstable landfill environments (Lutton, Regan, and Jones, 1979).

Another disadvantage to capping is the high cost of proper soil and drainage materials in certain areas of the country. However, these high costs would seldom result in selection of the excavation and removal alternative on the basis of economics. The most probable reason for selecting against capping at a site with extensive subsurface contamination would be an unacceptable risk to a source of drinking water where even groundwater monitoring would not offer enough assurance that severe contamination would not occur. Such a situation, for example, may involve an extremely leachable and highly toxic contaminant.

3.1.3 Design Consideration

The primary purpose of a cap is to minimize contact between infiltrating rainwater and the emplaced wastes. There are two basic designs: multilayered and single-layered. Of these, the multi-layered caps are the most common and are required by the RCRA land disposal regulations of 40 CFR, Subparts K through N. However, a single-layered cap may be acceptable when a site is being temporarily covered; in an area where evapotranspiration far exceeds rainfall and there is little or no groundwater; or when there is absolute assurance that the integrity of such a cap will be continually maintained. It should be noted that a gas collection system should always be included in the design of a cover when there is any indication that the underlying wastes may generate gas. The design considerations of the multi-layered and single-layered caps are discussed separately below.

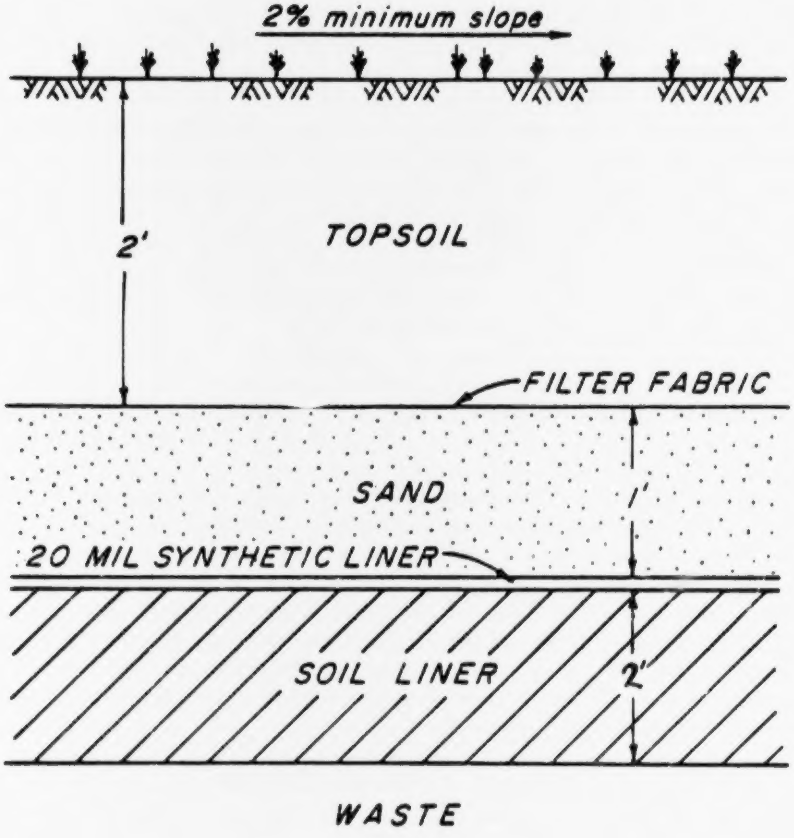
3.1.3.1 Multi-layered Caps

The design of multi-layered caps generally conforms to EPA's guidance under RCRA which recommends a three-layered system consisting of an upper vegetative layer, underlain by a drainage layer over a low permeability layer (USEPA, 1982). Figure 3-1 provides an example of a multi-layered cap. The vegetative layer in Figure 3-1 is served by the topsoil layer; the drainage layer is composed of sand; and the low permeability layer is formed by the combined synthetic and soil liner system. The cap functions by diverting infiltrating liquids from the vegetative layer through the drainage layer and away from the underlying waste materials.

The low permeability layer of the multi-layered cap can be composed of natural soils, admixed soils, a synthetic liner, or any combination of these materials. However, a synthetic liner overlying at least 2 feet of low permeability natural soil or soil admix is recommended because the synthetic liner allows virtually no liquid penetration for a minimum of 20 years, while the soil layer provides assurance of continued protection even if the synthetic liner fails.

Standard design practices specify permeabilities of less than or equal to 10^{-7} cm/sec for the soil liner (Cope et al., 1984). This specification would require a natural soil in the CL/CH range of the Unified Soil Classification System (USCS) (not less than 50 percent by weight passing a No. 200 sieve); however, blending of different on-site soil types can broaden the grain size distribution of a soil and minimizes its infiltration capacity. Well-graded soils are less permeable than those with a small range of grain sizes, and mixing of local coarse and fine-grained soils is a cost-effective method of creating stronger and less porous cover soil (Lutton, Regan, and Jones, 1979).

FIGURE 3-1.
MULTIMEDIA CAP



When sufficient fine grained soils are not available to achieve the desired permeability, clay material can be brought in. Bentonite, a natural clay with high swelling properties, is often transported to a site and mixed with on-site soil and water to produce the low permeability layer of the cap. Blending can often be accomplished in place using a blade or harrow to turn and mix the soil to suitable depths (Lutton, Regan, and Jones, 1979).

Chemical stabilizers and cements can be added to relatively small amounts of on-site soils to create stronger and less permeable surface sealants. Portland cement or bitumen (emulsified asphalt or tar) is suitable for mixing with sandy soils to stabilize and waterproof them. Site-specific mixing, spreading, and compacting procedures are required. For a soil-cement, approximately 8 percent (by weight) dry cement is blended into the soil with a rotary hoe or tiller as water is added. Intermittent sprinkling over several days may be required before compaction and solidification are achieved (Lutton, Regan, and Jones, 1979).

Soils may also be treated with lime, fly ash, bottom ash, and furnace slag, however, these materials should be tested for hazardous metals and organics prior to use. These materials contribute pozzolanic (cementing) properties to the resulting mixture, optimize the grain size distribution, and reduce shrink/swell behavior. Lime applied as 2 to 8 percent (by weight) calcium oxide or hydroxide is suitable for cementing clayey soils. Sands and gravels are more suitable for combined lime-fly ash treatment than are finer-grained soils (Lutton, Regan, and Jones, 1979). Rotary tiller mixing followed by water addition and compaction is the general application sequence for these mixtures. If a synthetic liner is present, liner life may be prolonged by lime addition to supporting soil (Fields and Lindsay, 1975). Other soil additives include chemical dispersants and swell reducers. Soluble salts such as sodium chloride, tetrasodium pyrophosphate, and sodium polyphosphate are added primarily to fine-grained soils with clay minerals to deflocculate the soils, increase their density, reduce permeability, and facilitate compaction. Additives are more effective with montmorillonite clay than with kaolinite or illite.

Flexible synthetic membranes are made of polyvinyl chloride (PVC), chlorinated polyethylene (CPE), ethylene propylene rubber, butyl rubber, Hypalon® and neoprene (synthetic rubbers), and elasticized polyolefin. Synthetic liners are generally more expensive and labor-intensive sealing materials that require special field installation methods. Thin sheets are available in sections of variable width and the sheets are overlain and spliced in the field (according to manufacturer's specifications). Special adhesives and sealants are used to ensure liner integrity. The chemical resistance of a cap synthetic liner is not usually critical. However, the potential for organic and/or corrosive vapors should be carefully evaluated before dismissing the resistance factor. The thickness and flexibility of a cap synthetic liner are crucial and should be carefully researched during the material selection process. Cope et al. (1984) and Matrecon, Inc. (1983) describe several important considerations for selecting synthetic liners. The slope of the low permeability layer should be between 3 and 5 percent to prevent erosion (if the upper synthetic layer fails) and pooling of rainwater. The underlying base of this layer should consist of fine to medium grade fill which will support the weight of the entire cap and not abrade the liner. If a clay or similarly fine grained soil liner is to be used, the underlying base must be sufficiently fine to preclude piping of the liner. Piping occurs when sections of an overlying fine-grained soil layer erode and fall into an underlying coarser grained soil layer (Matrecon, Inc., 1983). Piping may be prevented by placement of a suitably fine-meshed filter fabric between the two layers.

The drainage layer of the multi-layered cap is placed directly above the low permeability layer. The permeability of the drainage layer should be sufficiently high that it minimizes contact of infiltrating rainwater with the low permeability layer (Lutton, 1982). Current designs generally specify a material with greater than or equal to 10^{-3} cm/sec permeability (Cope et al., 1984). This layer can be composed of a sand in the SW or SP range of USCS (less than 5 percent passing through a number 100 sieve) or a coarser material. The thickness of the drainage layer depends on the amount of settling expected and the maximum volume of water that could enter it. The vegetative layer of the multi-layer cap is placed above the drainage layer, usually with a layer of filter fabric in between to prevent piping. The vegetative layer usually exceeds 2 feet in thickness, but may be greater depending on the frost depth, the maximum depth of root penetration, and the rate of anticipated soil loss. The frost depth must not be allowed to reach the low permeability layer because freezing and thawing cycles could greatly increase its permeability. The selection of a vegetative cover should include consideration of root penetration, erosion potential, and competitive advantage over other plant species in the area. These factors can be determined by consulting with local botany professors. Erosion potential of the soil, however can occur even when the vegetative cover has good soil retaining capabilities. Therefore, it is recommended that the soil in the vegetative layer have an erosion rate of less than 2 tons per acre, per year, using the U.S. Department of Agriculture Universal Soil Loss Equation (USLE). This equation is written:

$$A = R \times K \times L \times S \times C \times P$$

Where: A = average spoil loss in tons/acre for time period used for R

quantitative:
R = rainfall and run-off erosivity index
K = soil erodibility factor
L = slope length factor
S = slope steepness factor

qualitative:
C = cover/management factor
P = supporting practices factor

Directions for determining variables are given in Lutton, Regan, and Jones (1979) pp. 127-133. For information regarding soil sampling and testing, for local data on soils and climate, or for any form of technical assistance regarding selection of cover materials, regional and county Soil Conservation Services (SCS) offices should be consulted.

3.1.3.2 Single-Layered Caps

Single-layered caps can be constructed of any of the low permeability materials mentioned above. However, natural soil and admixes are not recommended because they are disrupted by freeze/thaw cycles and exposure to drying causes them to shrink and crack (Matrecon, Inc., 1983). The most effective single layer caps are composed of concrete and or bituminous asphalt. The thickness of these liners is dependent on the amount of anticipated settlement and the local weather conditions. Periodic application of special surface treatments for asphalt and concrete liners can greatly improve their life and effectiveness. It should be noted that single layered caps will not usually be acceptable unless there are extreme circumstances. For example, an asphalt cap that can be inspected on a frequent basis may be acceptable. Similarly, a temporary cap constructed of clay or natural soil may be used, depending on the length of time before a final remedial response is completed. Another potential opportunity to use a single-layered cap may arise in an area where evapotranspiration greatly exceeds precipitation and/or there is a great distance between the waste and the nearest source of usable groundwater. In these cases it may be acceptable to use an extremely low permeability soil or admix buried by natural soils beneath the frost penetration depth. The overlying soils would also protect the cap from drying and cracking.

3.1.4 Construction/Implementation Considerations

This section discusses construction considerations for multi-layered caps. Construction considerations for single-layered caps vary depending on the cap materials used (e.g., concrete, asphalt, clay), therefore, appropriate construction guidance should be acquired according to the cap material being considered. The EPA document entitled Lining of Waste Impoundment and Disposal Facilities (Matrecon, Inc., 1983) contains references for constructing caps out of several different materials.

The first layer of a multi-layered cap is the foundation layer. It should be composed of soil materials that are structurally capable of supporting the weight of the cap. Tests that may be used in evaluating this layer include unconfined compressive strength tests (ASTM D2166), triaxial compression tests (D2850), direct shear tests (ASTM D3080), and grain size analyses (ASTM D421, D422, and D1140). The foundation material should be spread over the wastes in 6-inch lifts and compacted to its maximum achievable density (ASTM D698 and D1557). The structural stability tests mentioned above should be run on each lift in sufficient number to assure uniformity. The final shape of this layer should be the same as the final design shape. If the foundation layer is also intended to be the bedding layer for the overlying low permeability layer, it should meet grain size specifications contained in Evaluating Cover Systems for Solid and Hazardous Waste (Lutton, 1982). Otherwise, an appropriately sized filter fabric should be used between the foundation and low permeability layers.

The low permeability layer should be placed in 6-inch lifts and compacted with a bulldozer, a sheepsfoot roller, or other heavy equipment to over 90 percent of its dry density at its optimum water content as determined by ASTM D1557 or another suitable test method (Ehrenfelder and Bass, 1983). Permeability should be measured at the completion of each lift at a rate of at least 2 tests per acre. Compaction can be accomplished with a bulldozer or a sheepsfoot roller. Permeability and density tests will determine the minimum weight of the compaction equipment and the number of passes needed. The selection of materials for the low permeability layer should consider all of the structural stability tests mentioned above and the Atterberg limits tests (ASTM D423, D424, D427, and D2217) (USEPA, 1983). These limits should be as wide as possible to prevent future increases in permeability due to drying or wetting of the cap. The thickness of the low permeability zone should be at least 2 feet, but should be increased if a lot of settling is expected in the underlying wastes.

A synthetic liner should be placed and seamed according to manufacturers specifications. This liner should be at least 20 mils thick and the material and seams should be tested for peel adhesion (ASTM D413, Method 1 and D1876) and shear strength (ASTM D816, Method B, modified and D882, Method A modified) (Cope et al., 1984). Additional tests on the liner should be conducted as recommended by the liner installation company.

The drainage layer should have a permeability of 10^{-3} cm/sec or greater (USEPA, 1982). This layer should be placed in 6-inch lifts and should be at least 1 foot thick. A thicker liner should be used if more than a few inches of settling or subsidence is expected in the underlying waste. If it is placed directly over the synthetic liner, the drainage layer material must be free of sharp objects that might puncture the liner. Sand classified as SP by the USCS would fulfill the liner bedding requirements.

Filter fabric should be placed above the drainage layer to prevent clogging from the overlying vegetative layer. This material is generally rolled in overlapping strips over the drainage layer in accordance with manufacturers specifications. The pore size of this layer should be large enough to allow proper drainage, but small enough to prevent soil migration into the drainage layer.

The vegetative layer should be at least 2 feet thick to accomodate expected root penetration, and it should be spread evenly and not overly compacted. The thickness of this layer should be greater than the deepest zone of frost penetration found in the area (USEPA, 1982). The vegetation should be a non-woody plant, preferably a grass, with almost no maintenance requirements once it is established.

3.1.5 Operation, Maintenance, and Monitoring

A final cap should be inspected on a regular basis for signs of erosion, settlement, or subsidence. It is recommended that inspections be conducted frequently in the first 6 months because problems are most likely to appear during this period (Lutton, 1982). Maintenance of the final cap should be limited to periodic mowing of the vegetation layer to prevent invasion by deep-rooted vegetation and burrowing animals. Any signs of unexpected settling or subsidence should be addressed immediately by removing the overburden to inspect and repair the affected areas.

3.1.6 Technology Selection/Evaluation

Capping is a reliable technology for sealing off contamination from the aboveground environment and significantly reducing underground migration of wastes. Caps can be constructed over virtually any site, and can be completed relatively quickly if the ground is not frozen or saturated. Most of the soil materials for capping are readily available in most areas of the country, and the synthetic materials are widely manufactured and distributed. The equipment used for implementing this technology is mostly standard road construction equipment, however some specialized testing equipment must be supplied by the liner installer or a soil testing company.

The performance of a properly installed, multi-layered cap is generally excellent for the first 20 years of service. However, after this time period, the integrity of the synthetic liner becomes uncertain and should be investigated regularly. Unforseen settling invasions by burrowing animals and deep-rooted plants also contribute to the need for periodic monitoring and maintenance of the cap. In addition, the groundwater monitoring wells, often associated with caps, need to be sampled periodically and maintained. However, these long-term maintenance requirements are usually considerably more economical than the alternative to a cap, which is excavation and removal of the wastes.

3.1.7 Costs

The cost of a cap depends on the type and amount of materials selected, the thickness of each layer, and the area of the country. General material and installation costs for caps larger than 400,000 square feet are presented in Table 3-2. In a recent RCRA Part B Permit Application for a 4-acre hazardous waste landfill the estimated installed cost of a multi-layered cap was $\$5.45/\text{ft}^2$ (SAIC, 1985). The design for this cap included 3 feet of top soil overlying a 1-foot sand layer overlying 1 foot of compacted clay overlying a 30-mil HDPE liner overlying 2 feet of compacted clay. Filter fabric was specified between the topsoil and sand drainage layer to prevent clogging. Quality control testing of each layer of the cap were included in the installation cost estimates.

TABLE 3-2.
1985 UNIT COSTS ASSOCIATED WITH CAPPING DISPOSAL SITES*

Element	Cost
Cleaning and Grubbing	\$1,100/acre
Excavation	\$1.60/yd^3
Earthfill	
berms and levees	\$2.10/yd^3
soil liners	\$3.10/yd^3
Backfill	\$3.10/yd^3
Soil Import	
drainage sand	\$10.50/yd^3
drainage rock (rounded)	\$10.50/yd^3
Soil placement	\$1.00/yd^3
Vegetation, mulch, and hydroseed	\$1,100/acre
Geotextile Fabrics	\$1.00 - \$3.10/yd^2
Bentonite Admix (2-9 lbs/yd^3)**	\$0.20 - \$1.10/ft^2
Membrane Liners	
Nonreinforced	
30 mil PVC	\$0.25 - \$0.35/ft^2
30 mil CPE	\$0.35 - \$0.45/ft^2
30 mil Butyl/EDPM	\$0.45 - \$0.55/ft^2
30 mil Neoprene	\$0.70 - \$0.80/ft^2
100 mil HDPE	\$1.10 - \$1.60/ft^2
Reinforced	
36 mil Hypalon (CSPER)	\$0.50 - \$0.60/ft^2
60 mil Hypalon (CSPER)	\$0.80 - \$1.00/ft^2
36 mil Hypalon	\$0.50 - \$0.60/ft^2
Installation, excluding earthwork	\$0.60 - \$1.20/ft^2

*Based on costs for a 400,000 ft^2 area presented in Cope et al., 1984, as updated by construction, labor, and material cost indices in Engineering News-Record 1983 and 1985.

**Includes mixing and placing.

PVC = polyvinyl chloride
CPE = chlorinated polyethylene
EDPM = ethylene-propylene-diene-monomer
CSPER = chlorosulfonated polyethylene (reinforced)

3.2 Floating Covers

3.2.1 General Description

Floating covers consist of a synthetic lining placed in one piece over an impoundment, with proper anchoring at the edges, and with floats to prevent the lining from submerging. This technology is used mainly to cover drinking water supply reservoirs, but it can be used temporarily to prevent overtopping of a waste lagoon prior to final closure.

3.2.2 Applications/Limitations

Floating covers cannot be considered as a final remedial action at a site, but they can be more cost effective than pumpdown and treatment to prevent overtopping of lagoon wastes. Covers are especially advantageous when a year or more will elapse before final closure of the lagoon. Floating covers shold not be considered on lagoons with weak berms or those located in areas that cannot support the weight of heavy construction equipment. Also, available cover materials may not always be compatable with the lagoon wastes.

3.2.3 Design Considerations

There are two basic floating cover designs. One design is based primarily on a rainwater collection sump which forms around the perimeter of the cover during rain events (Dial, Habegger, and Kays, 1979). Rainwater is directed toward this sump by various configurations of foam floats that are inserted and sealed into special pockets in the cover. The most common float configuration consists of one large center float with smaller laterial floats attached perpendicularly at about 36-foot centers (Figure 3-2). After the sump at the perimeter of the cover fills with water, it is drained either by random pumping with a portable pump, or by installing a flexible perforated hose in the area of the fold that is formed by the collected water (Kays, W.B. Globe Linings, Inc., Long Beach, CA, personal communication, 1984).

The other design consists of channels in the middle of the cover created by strings of segmented sand-filled tubes which are held at a constant depth by floats on either side of the channel (Burke et al., 1976). The sand-filled tubes are connected in parallel to flexible perforated collection tubes above and to the cover underneath. Rainwater, after striking the cover, drains through openings in the floats which line the collection channels, and is pumped off the cover through the collection tubes (Figure 3-3). The dimensions for each component of the system are determined through a few simple equations that are based on the dimensions of the area to be covered (Brown, S., Burke Rubber Co., San Jose, CA, personal communication, 1983).

FIGURE 3-2.
SCHEMATIC PLAN AND CROSS-SECTION OF A FLOATING COVER INCORPORATING THE PATENTED BURKE DESIGN

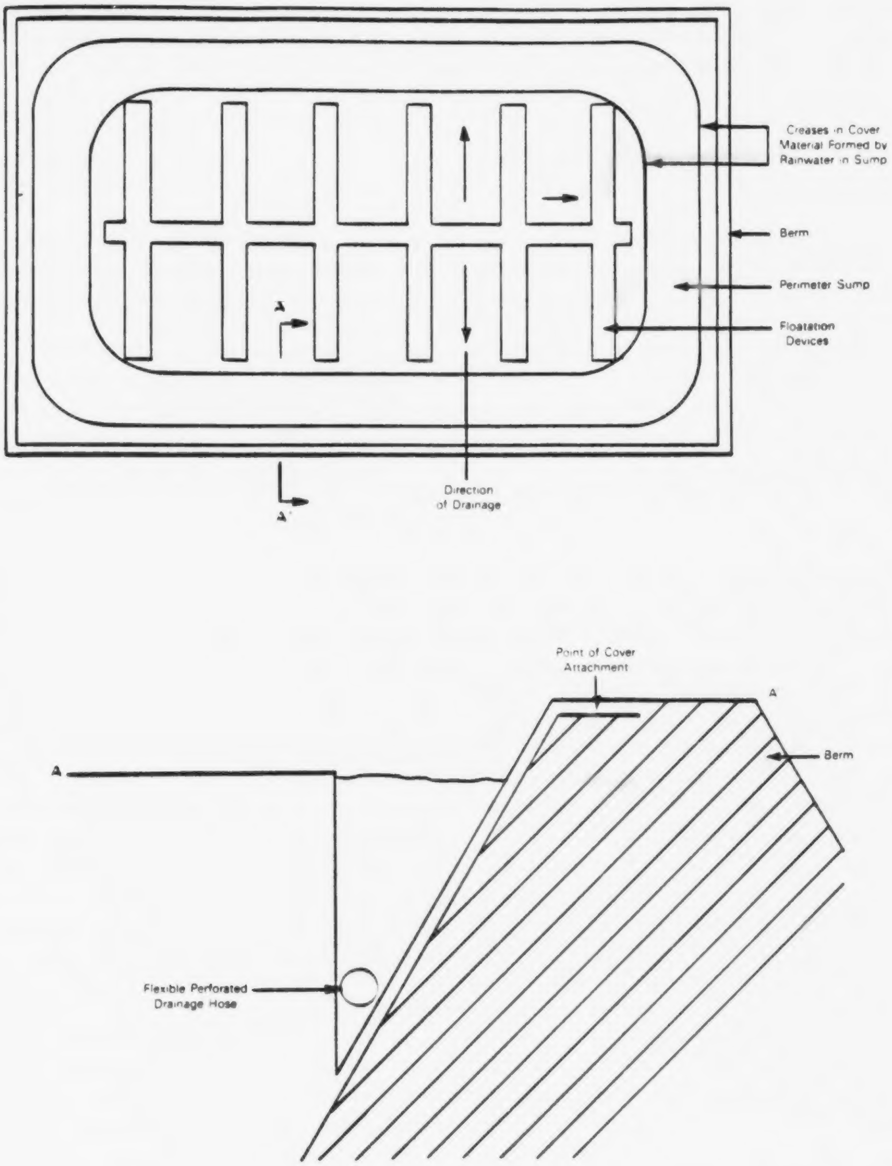
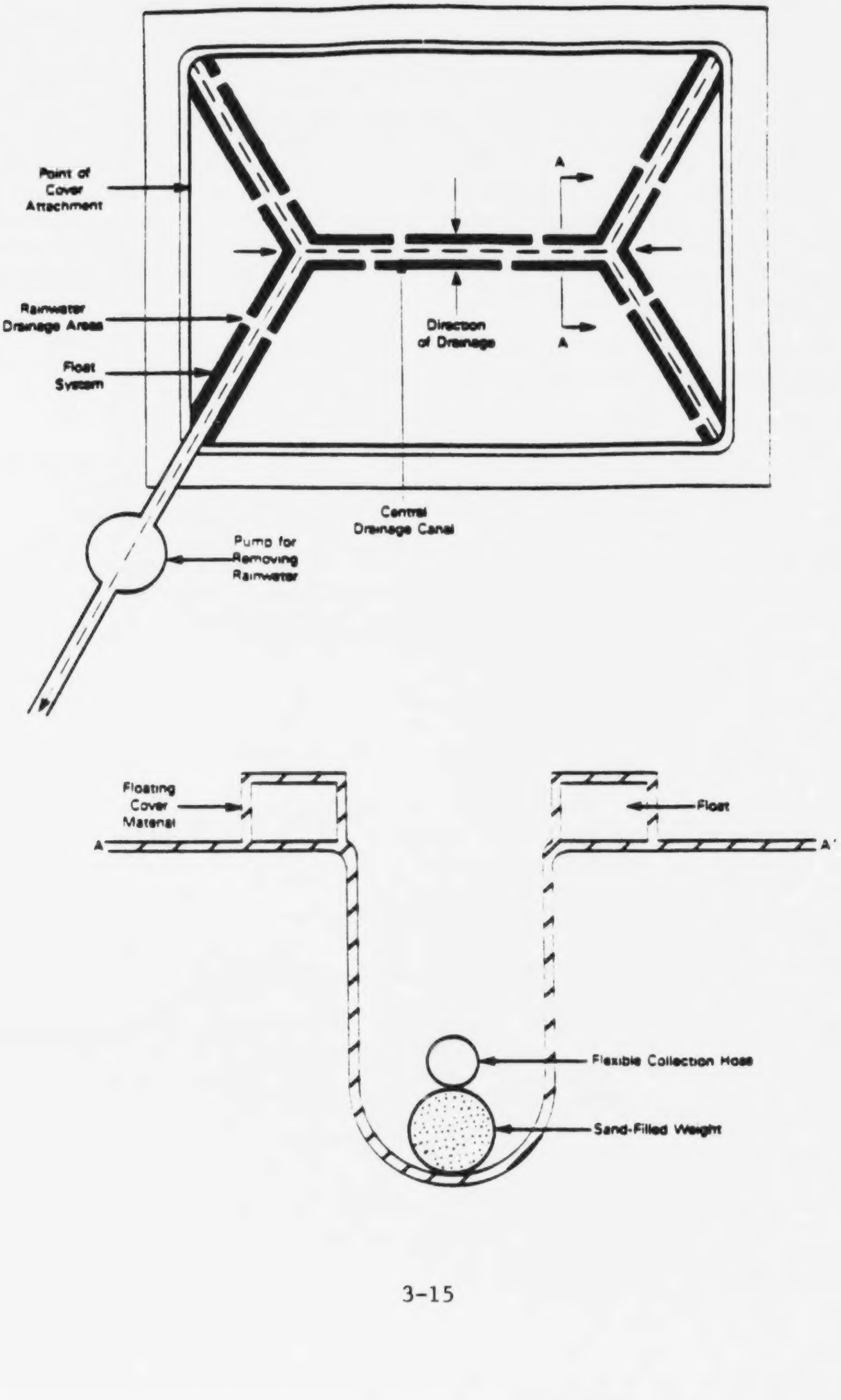

Creases in Cover Material Formed by Rainwater in Sump
Berm
Perimeter Sump
Floatation Devices
A
A'
Direction of Drainage
Point of Cover Attachment
A'
Berm
A
Flexible Perforated Drainage Hose

FIGURE 3-3.
SCHEMATIC PLAN AND CROSS-SECTION OF A FLOATING COVER INCORPORATING THE PATENTED BURKE DESIGN



The synthetic lining material for constructing a floating cover usually consists of 36-mil or 45-mil thick, reinforced Hypalon®, chlorinated polyethylene (CPE), or XR-5. Ethylene-propylene-diene-monomer (EPDM) and butyl rubber are not often used because of their tendancy toward cracking and their low field seaming quality. The chemical resistance of various lining materials is determined on a case-by-case basis through immersion testing (see Section 3.2.4).

If gases are expected to be a problem at a waste lagoon, a gas collection system can be included in the cover design. The gas collection system consists of several air chambers formed at various points along the cover floatation system (Figure 3-4). Each chamber is covered by the floating cover material and is equiped with openings which allow gases to flow up into the chamber from a triangular-shaped space formed between the cover, the covered liquid, and the side of the float. The upper end of the air chamber is connected to a short standpipe and then to a manifold pipe to allow pumping and collection of gases (Kays, 1976).

3.2.4 Construction/Implementation Considerations

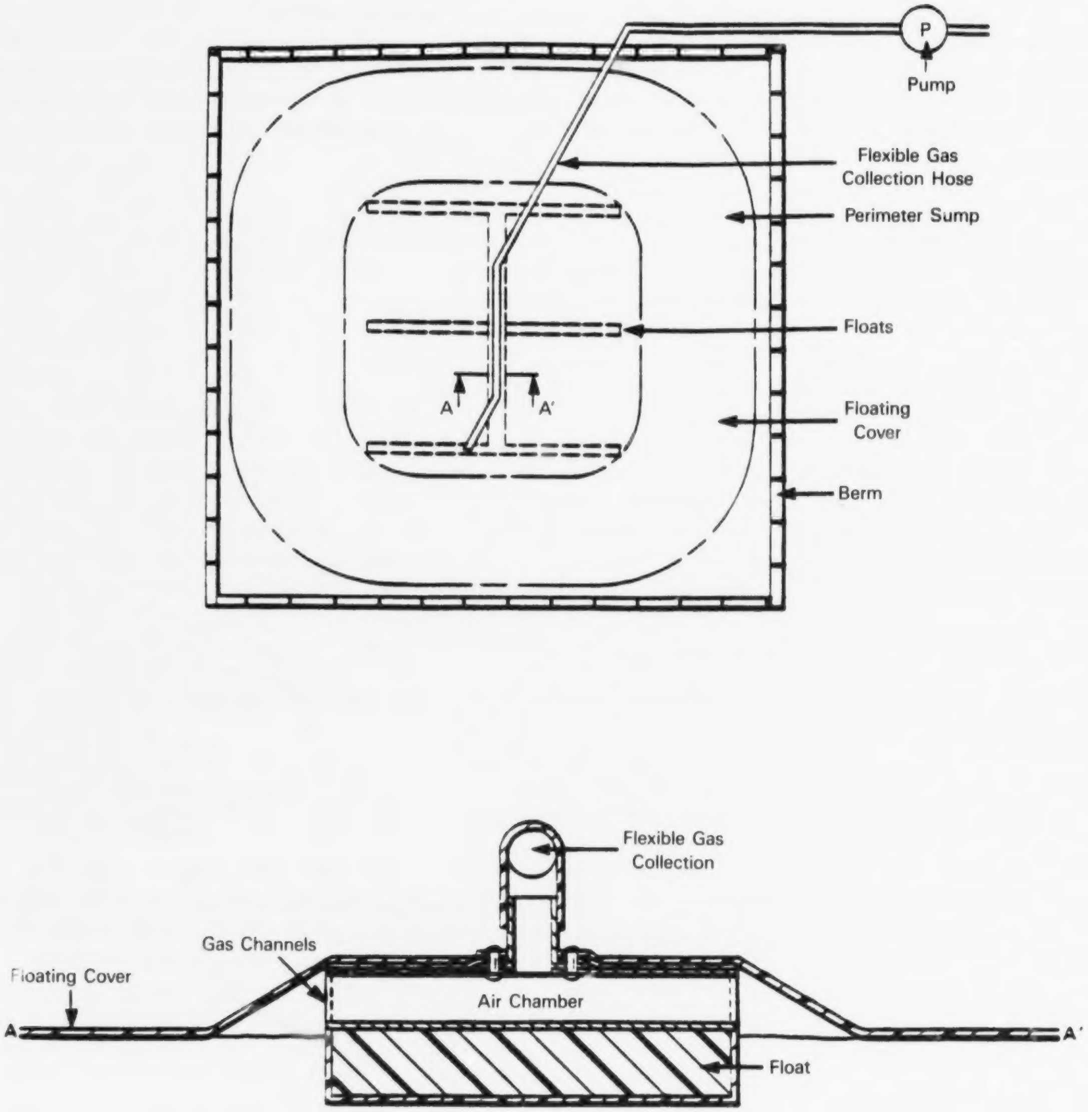
Sections of cover materials are shipped to the site on large wooden pallets. All pockets for insertion of floatation devices are generally made at the fabrication plant prior to shipment. Once at the site, each pallet is placed as near as possible to where that section of cover will be seamed into place. Cover materials are accordian-folded along their length and width for easy deployment. If the area to be covered is 20,000 square feet or less, and Hypalon® is being used, only one section of cover is required (Shuey, R., Palco Linings, Inc., South Plainfield, NJ, personal communication, 1983).

Installation techniques vary depending on whether the area to be covered is full of liquid or empty. An empty reservoir is easier to cover because the cover can be seamed together as it lies on the bottom of the reservoir.

To cover a reservoir which is not empty, pallets of cover material are prepared according to the estimated surface area of the bottom of the reservoir (Shuey, R., Palco Linings, Inc., South Plainfield, NJ, personal communication, 1984). For large reservoirs, the first row of pallets are placed at intervals along the edge of the reservoir so that they can be easily seamed end-to-end. Next, cables are attached along the leading edges of the joined cover segments so that the cover section can be dragged across the liquid. Before dragging the entire cover section into the reservoir, a new set of pallets are unloaded, and the next cover sections are unfolded and seamed onto the first. This process continues until all the cover sections are in place. Floats are inserted into their pre-formed pockets which are then seamed shut (Kays, W.B., Globe Linings, Inc., Long Beach, CA, personal communication, 1983; Shuey, R., Palco Linings, Inc., South Plainfield, NJ, personal communication, 1984).

Anchoring for cover systems on reservoirs follows two basic patterns. In one anchoring method, a concrete curb is poured around the reservoir. The

FIGURE 3-4.
SCHEMATIC PLAN AND CROSS-SECTION OF A PATENTED GLOBE FLOATING COVER AND GAS COLLECTION SYSTEM DESIGN



curb has 1/2-inch diameter bolts on 12-inch centers which are fitted into pre-cut, reinforced holes in the cover fabric and then through holes in metal battens (Kays, 1977). These battens are bolted down to produce a continuous seal around the perimeter of the cover. The other, less expensive, method for anchoring a cover begins by digging a 12-inch wide by 18-inch deep trench around the reservoir (Shuey, R., Palco Linings, Inc., South Plainfield, NJ, personal communication, 1984). Next, about 40 inches of the liner are placed into the trench so that it conforms with the sides and bottom. The trench is then backfilled and compacted with heavy equipment. On large reservoirs, these anchor systems should be augmented with a mechanism to counteract the effects of strong winds. One such method is achieved by stringing cables from the cover to the bottom of the reservoir at regular intervals (Dial, Habegger, and Kays, 1979). In this design, the stress from wind is shared between the perimeter anchor and the bottom anchor.

The selection of floating cover material is usually based on immersion tests performed by the cover installation company. The actual tests vary depending upon the installation company, however, some standard synthetic liner tests are described in USEPA (1983) and in Cope et al. (1984).

3.2.5 Operation, Maintenance and Monitoring

Floating covers should be inspected periodically for wearing, discoloration, tearing, and rainwater accumulation. Some covers must be manually pumped off after rain events; others have automatic pumping systems which need to be inspected. Any signs of tearing or other weaknesses should be repaired immediately upon detection.

Accumulation of snow and ice on floating covers is not a problem, and attempted removal of these materials could actually damage the cover. Stones, sticks, leaves, and other debris, however, should not be allowed to accumulate.

3.2.6 Technology Selection/Evaluation

Floating covers should be considered whenever there is a need to temporarily prevent overtopping in a waste lagoon prior to final closure. Although floating covers have not been used on many waste lagoons, there is strong evidence of their potential performance from successful applications at hundreds of potable drinking water reservoirs. One cover, placed over a reservoir in Fort Saint John, British Columbia, withstood the forces of 4 feet of ice and -40° F temperatures for 2 months (Kays, W.B., Globe Linings, Inc., Long Beach, CA, personal communication, 1984). Two other covers in Mobile, Alabama have been unaffected by winds up to 150 mph (Sahol, J.R., Globe Linings, Inc., Houston, TX, personal communication, 1983).

3.2.7 Costs

The installed costs of floating covers generally range from $2.50 to $3.50/square foot installed (Kays, W.B., Globe Linings, Inc., Long Beach, CA, personal communication, 1984). This cost range is based on a liner material cost of about $0.70/square foot for 36-mil reinforced Hypalon® or CPE. If a more chemically resistent material were required, material costs could reach $1.50/square foot. Additional costs of up to $2.00/sqaure foot could be incurred if removal of vegetation were required or if difficult engineering or site access problems were present at the proposed site. The incorporation of a gas collection system into the floating cover will increase the installed price by appoximately $1.00/square foot (Kays, W.B., Globe Linings, Inc., Long Beach, CA, personal communication, 1984).

3.3 Grading

3.3.1 General Description

Grading is the general term for techniques used to reshape the surface of covered landfills in order to manage surface water infiltration and run-off while controlling erosion. The spreading and compaction steps used in grading are techniques practiced routinely at sanitary landfills. The equipment and methods used in grading are essentially the same for all landfill surfaces, but applications of grading technology will vary by site. Grading is often performed in conjunction with surface sealing practices and revegetation as part of an integrated landfill closure plan.

3.3.2 Applications/Limitations

Regrading is a relatively inexpensive remedial action component when suitable cover materials are available on-site or close to the disposal site.

The techniques and equipment used in grading operations are well established and are widely used in all forms of land development. It is usually possible to find contractors and equipment locally, thus expediating the work and avoiding extra expenses.

Surface grading serves several functions:

- Reduces ponding which minimizes infiltration and reduces subsequent differential settling
- Reduces runoff velocities to reduce soil erosion
- Roughens and loosens soils in preparation for revegetation

- Can be a factor in reducing or eliminating leaching of wastes.

There may, however, be certain disadvantages associated with grading the surface of a given site. Large quantities of a difficult to obtain cover soil may be required to modify existing slopes. Suitable sources of cover material may be located at great distances from the disposal site, increasing hauling costs. Also, periodic regrading and future site maintenance may be necessary to eliminate depressions formed through differential settlement and compaction, or to repair slopes that have slumped or become badly eroded (Tolman et al., 1978).

3.3.3 Design Considerations

Grading techniques modify the natural topography and run-off characteristics of waste sites to control infiltration and erosion. This is accomplished by establishing continuous surface grades to ensure that runoff water does not pond. The choice of specific grading techniques for a given waste disposal site will depend on desired site-specific functions of a graded surface. A graded surface may reduce or enhance infiltration and detain or promote run-off.

Erosion control may be considered a complicating variable in the design and performance of a grading scheme. The design of graded slopes at waste disposal sites should balance infiltration and run-off control against possible decreases in slope stability and increases in erosion. The design of specific slope configurations, the choice of cover soil type, the degree of compaction, and the types of grading equipment used will all depend on local topography, climate, future land use of the site, and drainage methods.

Improperly graded slopes may deform or fail, opening cracks, exposing wastes, and allowing lateral seepage of leachate. Soils used to cover graded slopes should be selected on the basis of shear strength and erodibility (Lutton, Regan, and Jones, 1979). Soils high in silt and fine sand and low in clay and organic matter are generally most erodible (USEPA, 1976). Also, the longer and steeper the slope is, and the sparser the vegetation cover, the more susceptible it is to erosive forces.

Manipulation of slope length and gradient is the most common grading technique used to reduce infiltration and promote surface water runoff. A slope of at least 5 percent, but not greater than approximately 18 percent, is recommended as sufficient to promote runoff and decrease infiltration without risking excessive erosion (Lutton, 1978). More specifically, the center of the disposal site should be the highest elevation, with top surface slopes graded 6 to 12 percent and side slopes no steeper than 18 percent. This grading will enhance run-off and minimize infiltration and most soils will remain stable on such slopes (Tolman et al., 1978). For equal surface areas of land, doubling the slope length will increase soil losses by 1.5 times (USEPA, 1976). Therefore, where off-site transport of contaminated soil due to water erosion is a major consideration, the length of graded slopes should be minimized. Minimization of slope length can be accomplished by terracing, as discussed in Section 3.5.3.

Grading is often accompanied by other water handling measures such as sumps and sewers, as well as treatment systems for runoff from an uncontrolled site.

3.3.4 Construction Considerations

It is important that grades be maintained as accurately as possible. Reverse grades should not be permitted. Also, to provide a firm subgrade and prevent seal failure, existing cover material should be compacted to a Proctor density of 70 to 90 percent of maximum (Tolman et al., 1978). This can be achieved through repeated passes of leveling equipment over the area in several different directions.

All states and localities publish guidelines and regulations for performance of any grading work. These publications should be consulted before a grading system is designed.

Grading methods used to roughen and loosen soils in preparation for revegetation can be classified as scarification, tracking, and contour furrowing. These methods actually detain run-off and promote infiltration in order to establish vegetation.

Scarification can be accomplished by harrowing along the ground contour or by dragging the bucket teeth of a front-end loader over the ground. Specially equipped crawler tractors can also perform scarification. Tracking is performed on steep slopes where movement along the contour is not feasible. A cleated crawler tractor is run up and down the slope, leaving shallow grooves that run parallel to the contour. Contour terracing or furrowing is used on long slopes in conjunction with other roughening techniques to disrupt and slow surface runoff. This effect is accomplished by running a bulldozer parallel to the contour. Dirt is allowed to dribble off the blade end, creating small depressions that interrupt downslope surface water flow (USEPA, 1976).

The equipment types used to construct graded slopes consist of both standard and specialized landfill vehicles. Excavation, hauling, spreading, and compaction of cover materials are the major elements of a complete grading operation. Landfill grading vehicles include scrapers, crawler dozers and loaders, rubber-tired dozers and loaders, and landfill compactors (Figure 3-5). Table 3-3 summarizes the cover material handling capabilities of these equipment types.

Crawler tractors (dozers and loaders) are excellent landfill grading machines (Brunner and Keller, 1972). Dozers are fitted with U-shaped landfill blades for pushing and spreading; loaders can be fitted with landfill blades and multipurpose buckets for excavation and spreading.

FIGURE 3-5.
GRADING VEHICLES AND ACCESSORIES

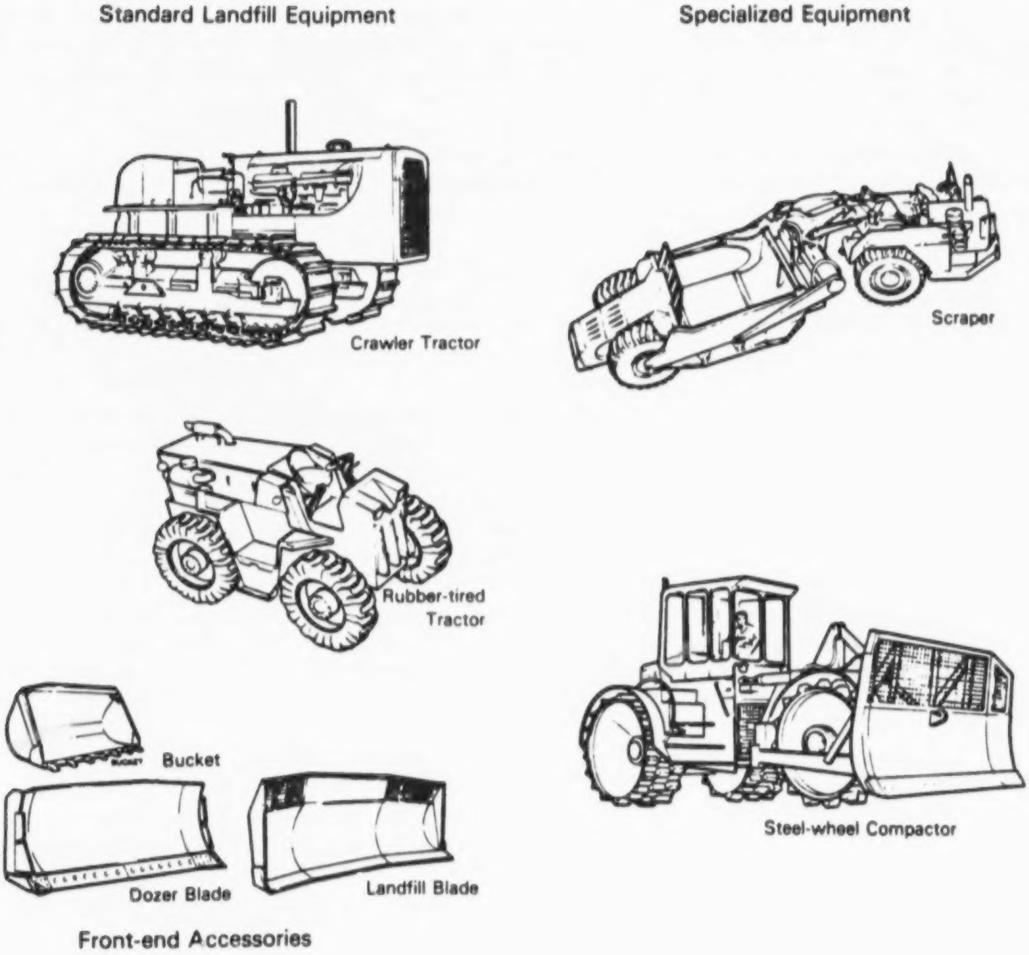


TABLE 3-3.
COVER MATERIAL HANDLING CHARACTERISTICS OF LANDFILL EQUIPMENT[1]

	Rating[2]			
Equipment	Excavating	Spreading	Compacting	Hauling
Crawler dozer	E	E	G	NA
Crawler loader	E	G	G	NA
Rubber-tired dozer	F	G	G	NA
Rubber-tired loader	F	G	G	NA
Landfill compactor	P	G	E	NA
Scraper	G	E	NA	E

[1] Assuming easily workable soil and cover material and haul distance 1,000 ft.

[2] Rating key: E, excellent; G, good; F, fair; P, poor; NA, not applicable.
Source: Brunner and Keller, 1972

Rubber-tired dozers and loaders are generally faster and more agile than crawler machines; however, they do not excavate as well. Rubber-tired dozers are used infrequently at landfill sites, because they do not grade as well as crawler dozers on the irregular and spongy surface provided by compacted refuse (Brunner and Keller, 1972).

Specialized landfill vehicles include compactors and scrapers. Steel-wheeled landfill compactors are excellent machines for spreading and compacting on flat to moderate slopes. Scrapers are effective in excavating, hauling, and spreading cover materials over relatively long distances (Brunner and Keller, 1972).

3.3.5 Operation, Maintenance, and Monitoring

As all fill material has the tendency to settle, periodic monitoring of the site is necessary. Smoothing operations may be required for several years as settling occurs.

All seeps and ponds should be expediantly corrected. Repeated occurrences may indicate failure of the site cover and should be promptly investigated.

It is important upon completion of grading to establish a vegetation cover as quickly as possible (revegetation is discussed in Section 3.4). This cover is essential to help prevent drying and erosion.

3.3.6 Technology Selection/Evaluation

Surface grading of covered disposal sites, when properly designed and constructed to suit individual sites, can be an economical method of controlling infiltration, diverting run-off, and minimizing erosion. A properly sealed and graded surface will aid in the reduction of subsurface leachate formation by minimizing infiltration and promoting erosion-free drainage of surface run-off. Grading can also be used to prepare a cover soil capable of supporting beneficial plant species.

Estimated total costs for grading reported in the cost compendium (USEPA, 1985) (and updated to 1985 dollars) ranged from a low of $4150/acre to $16,814/acre. The lowest grading cost estimate reflected the cost of on-site hauling, spreading and compacting of a one-foot thick soil layer and a 6-inch sand layer. This estimate assumes the use of on-site soil. The highest estimate reported in the cost compendium also involved the use of on-site soil but included excavation and grading costs for on-site soil. Additional costs (30 percent) were included in these estimates to cover overhead and a contingency allowance.

1985 Operating and Maintenance costs for a graded area have been estimated to include the following (USEPA, 1985):

Item	Cost
Annual inspection	$518/year
Mowing/revegetation	$622/year
Erosion control and drainage maintenance	$207/year
Repairs resulting from shrink/swell or freeze/thaw forces	$207/year

Grading is considered essential to the continued performance and reliability of a cap. The performance and reliability of a graded surface depends upon effective revegetation. Grading is easy to implement and can generally be performed by local contractors. Grading rarely poses safety hazards to field personnel when performned on a properly capped site. However, when grading is performed to improve site drainage at an uncontrolled site, it may pose a risk to worker safety where drums and explosives are buried near the surface.

3.3.7 Costs

Unit costs associated with grading equipment and methods are presented in Table 3-4. Costs associated with heavy equipment maintenance (fuel, repairs, etc.) are not addressed. Costs of excavation, hauling, spreading and compaction will vary depending on equipment type and size, type of cover material

TABLE 3-4.
1985 UNIT COSTS ASSOCIATED WITH GRADING COVERED DISPOSAL SITES

Description	Unit Cost	Source
Topsoil (sandy loam) from borrow pits, excavation, haulng, spreading, and grading (within 25 miles); labor, materials, and equipment	$15/yd^3	1
On-site excavation, hauling, spreading, and compaction of earth (1,000-5,000 ft haul); labor and equipment only	$2.02-3.86/yd^3	1
Sandy loam topsoil; material only	$2.25/yd^3	1
Excavate, haul 2 miles, spread and compact loam, sand, or loose gravel (with front-end loader); labor, material, and equipment	$6.65-6.72/yd^3	1
Grading, site excavation, and fill (no compaction);		2
75 hp dozer, 300 ft haul	$3.39/yd^3	
300 hp dozer, 300 ft haul	$2.34/yd^3	
Field density compaction testing of soils	$104/day	1

[1] McMahon, 1984

[2] Godfrey, 1984

being graded, haul distance, support labor required, and unforeseen construction difficulties.

These costs are representative of average contractor bid prices for performance of the work, and therefore include charges for overhead and profit. All costs are reported in 1985 dollars.

3.4 Revegetation

3.4.1 General Description

The establishment of a vegetative cover is a cost-effective method to stabilize the surface of hazardous waste disposal sites, especially when preceded by capping and grading. Revegetation decreases erosion by wind and water and contributes to the development of a naturally fertile and stable surface environment. Also, the technique can be used to upgrade the appearance of disposal sites that are being considered for various re-use options. Short-term vegetative stabilization (i.e., on a semi-annual or seasonal basis) can also be used as a remedial technique for disposal sites.

A systematic revegetation plan will include: (1) selection of suitable plant species, (2) seedbed preparation, (3) seeding/planting, (4) mulching and/or chemical stabilization, and (5) fertilization and maintenance.

3.4.2 Applications/Limitations

Revegetation may be part of a long-term site reclamation project, or it may be used on a temporary or seasonal basis to stabilize intermediate cover surfaces at waste disposal sites. Revegetation may not be feasible at disposal sites with high cover soil concentrations of phytotoxic chemicals, unless these sites are properly sealed and vented and then recovered with suitable topsoil.

Grasses such as fescue and lovegrass provide a quick and lasting ground cover, with dense root systems that anchor soil and enhance infiltration. Legumes (lespedeza, vetch, clover, etc.) store nitrogen in their roots, enhancing soil fertility and assisting the growth of grasses. They are also readily established on steep slopes. Shrubs such as bristly locust and autumn olive also provide a dense surface cover, and certain species are quite tolerant of acidic soils and other possible disposal site stresses. Trees are generally planted in the later stages of site reclamation, after grasses and legumes have established a stable ground cover. They help provide long-term protective cover and build up a stable, fertile layer of decaying leaves and branches. A well mixed cover of grasses, shrubs, and trees will ultimately restore both economic and aesthetic value to a reclaimed site, providing suitable habitat for populations of both humans and wildlife. Table 3-5 summarizes the suitability of various grasses and legumes for revegetation

TABLE 3-5.
IMPORTANT CHARACTERISTICS OF GRASSES AND LEGUMES

Characteristic	Degree *	Common Examples
Texture	Fine	Kentucky bluegrass, bentgrass, red fescue
	Coarse	Smooth bromegrass, reed canarygrass, timothy
Growth height	Short	Kentucky bluegrass, buffalograss, red fescue
	Medium	Redtop, perennial ryegrass
	Tall	Smooth bromegrass, timothy, switchgrass
Growth habit	Bunch	Timothy, big bluestem, sand dropseed, perennial ryegrass
	Sod former	Quackgrass, smooth bromegrass, Kentucky bluegrass, switch grass
Reproduction	Seed	Red and alsike clover, sand dropseed, rye, perennial ryegrass, field bromegrass
	Vegetative	Prairie cordgrass, some bentgrasses
	Seed and vegetative	White clover, crownvetch, quackgrass, Kentucky bluegrass, smooth bromegrass
Annual	Summer	Rabbit clover, oats, soybeans, corn, sorghum
	Winter	Rye, hairy vetch, field bromegrass
Perennials	Short-lived	Timothy, perennial ryegrass, red and white clover
	Long-lived	Birdsfoot trefoil, crownvetch, Kentucky bluegrass, smooth bromegrass
Maintenance	Difficult	Tall fescue, reed canarygrass, timothy, alfalfa
	Moderate	Kentucky bluegrass, smooth bromegrass
	Easy	Crownvetch, white clover, birdsfoot trefoil, big bluestem
Shallow rooted	Weak	Sand dropseed, crabgrass, foxtail, white clover
	Strong	Timothy, Kentucky bluegrass
Deep rooted	Weak	Many weeds
	Strong	Big bluestem, switchgrass, alfalfa, reed canarygrass
Moisture	Dry	Sheep fescue, sand dropseed, smooth bromegrass
	Moderate	Crested wheatgrass, red clover
	Wet	Reed canarygrass, bentgrass
Temperature	Hot	Lehman lovegrass, fourwing saltbush, ryegrass
	Moderate	Orchard grass, Kentucky bluegrass, white clover
	Cold	Alfalfa, hairy vetch, smooth bromegrass, slender wheatgrass

* Variety, specific characteristic, subcharacteristic, or favored condition.

Source: Lutton, 1982

purposes. Native species of trees and shrubs, particularly those with a shallow root system, should generally be specified.

The selection of suitable plant species, the use of appropriate mulches and stabilizers, the application of required doses of lime and fertilizers, and optimum timing in seeding will help ensure the establishment of an effective vegetative cover. However, unforeseen difficulties may compromise the effectiveness of revegetation. Clays or synthetic barriers below supporting topsoil in poorly drained areas may cause swamping of cover soil and subsequent anaerobic conditons. A cover soil which is too thin may dry excessively in arid seasons and irrigation may be necessary. Improperly vented gases and soluble phytotoxic waste components may kill or damage vegetation. The roots of shrubs or trees may penetrate the waste cover and cause leaks of water infiltration and gas exfiltration. Also, periodic maintenance of revegetated areas--liming, fertilizing, mowing, replanting, or regrading eroded slopes--will add to the costs associated with this remedial technique.

3.4.3 Design Considerations

Long-term vegetative stabilization generally involves the planting of grasses, legumes, and shrubs. The establishment of short-term, seasonal vegetative cover is limited principally to species of grasses. The selection of suitable plant species for a given disposal site depends on several site-specific variables. These variables include cover soil characteristics (grain size, organic content, nutrient and pH levels, and water content), local climate, and site hydrology (slope steepness and drainage characteristics). Individual species must be chosen on the basis of their tolerance to such site-specific stresses as soil acidity and erodibility and elevated levels of landfill gases or phytotoxic waste components (e.g., heavy metals, salts) in cover soil. Other important considerations include the species compatibility with other plants selected to be grown on the site, resistance to insect damage and diseases, and suitability for future land use (USEPA, 1976).

Long-term vegetative stabilization and site reclamation require the proper planting of compatible mixes of grasses, legumes, shrubs, and trees. Short-term revegetation efforts generally require the use of low-cost, quick-growing perennial and self-seeding annual species, usually grasses (USEPA, 1976). In areas where a quick vegetative cover is essential to preventing erosion and pollutant transport, the use of an approved sod could prove beneficial.

A study of vegetative growth in landfill environs (Gilman et al., 1979) ranked the relative tolerance of selected tree species to high cover soil concentrations of carbon dioxide and methane, a typical landfill condition that causes the displacement of oxygen from the root zones of growing plants. The tree species included (in order of decreasing tolerance): Black Gum, Norway Spruce, Gingko, Black Pine, Bayberry, Mixed Poplar, White Pine, Pin Oak, Japanese Yew, American Basswood, American Sycamore, Red Maple, Sweet Gum, Green Ash, and Honey Locust. The same study found that mounds of topsoil underlain by clay gas-barriers, or trenches underlain with polyethylene sheeting and vented with perforated PVC vent pipes (Figure 3-6), effectively prevented the migration of landfill gases (products of anaerobic decomposition) into the root zone of trees. A further discussion on gas migration control can be found in Section 6. The Gilman study also concluded that woody plant species are more likely to survive on a completed fill if planted when small, generally less than three feet tall (Gilman et al., 1979).

Loamy topsoils--those with nearly equal percentages of clay, silt, and sand-sized grains--are generally best suited for revegetation establishment. They are easily seeded and allow easy root penetration. Sandy soils may be productive when blended or mulched with organic matter (Lutton, Regan, and Jones, 1979).

Mulches or chemical stabilizers may be applied to seeded soils to aid in the establishment of vegetative cover. Organic mulches such as straw, hay, wood chips, sawdust, dry bark, bagasse (unprocessed sugar cane fibers), excelsior (fine wood shavings), and manure protect bare seedbed slopes from erosion prior to germination. Also, thin blankets of burlap, fiberglass, and excelsior can be stapled down or applied with asphalt tacks to form protective mulch mats for germinating seedbeds.

Mulches conserve soil moisture, dissipate raindrop energy, moderate soil temperatures, prevent crusting, increase infiltraton, and generally control wind and water erosion. Stabilizers reduce soil water loss and enhance plant growth by temporarily stabilizing seeded soils against wind and water erosion. They can also be used to stabilize graded soils in the off season until spring seeding. Stabilizers are used extensively in arid regions to help dry, permeable soils retain soil moisture (USEPA, 1976).

Petroleum-based products such as asphalt and resins are often suitable and are frequently used as mulching materials. However, some of these materials are toxic and selection should be made only after careful evauation. Specially formulated emulsions of asphalt under various trade names have been used throughout the world to prevent erosion, reduce evaporation, promote seed germination, and warm the soil to advance the seeding date. The film clings to but does not deeply penetrate the soil. It is not readily destroyed by wind and rain and remains effective from 4 to 10 weeks. Application rates of 1000 to 1200 gallons/acre are usually required to control erosion. Asphalt mulches cost about twice the applied cost of a straw mulch (Lutton, 1982).

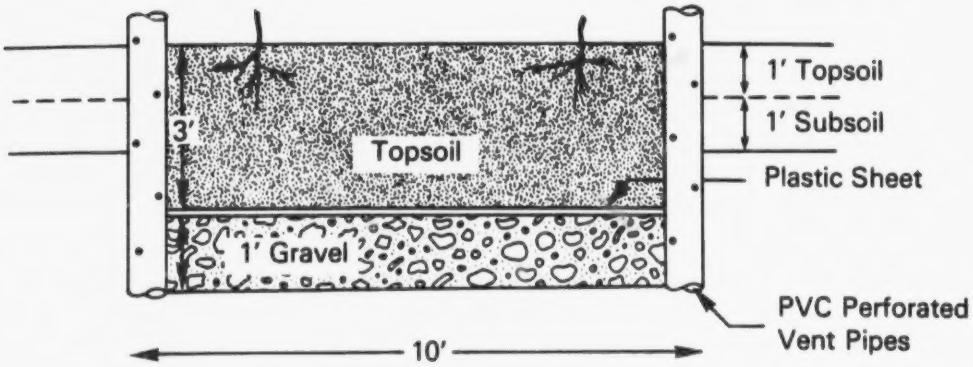
In field tests comparing the effectiveness of these chemical additives in controlling erodibility of several regional soil types in Virginia, none of the stabilizers tested were determined to be as cost-effective as conventional mulches of straw and asphalt-emulsions (Lutton, Regan, and Jones, 1979).

3.4.4 Construction/Implementation Considerations

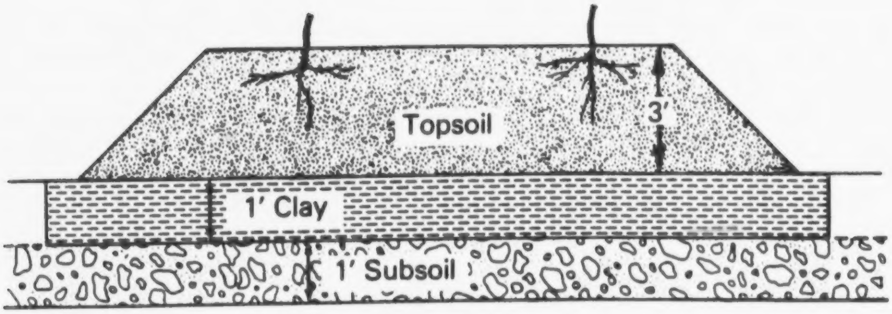
The optimum time for seeding depends on local climatic considerations and the individual species adaptations. For most perennial species in most localities, early fall seeding is recommended. Annuals are usually best seeded in

FIGURE 3-6.
GAS MIGRATION CONTROLS

Cross-Section End View of Gas Barrier Trench



Cross-Section End View of Soil Mound



Source: Gilman et al., 1979

spring and early summer, although they can be planted for quick vegetation whenever soil is damp and warm (Lutton, Regan, and Jones, 1979). In mild climates (e.g., southeastern United States) the growth of both summer and winter grasses will extend the range of evapotranspiration and erosion resistance for cover soils (Lutton, Regan, and Jones, 1979).

Seedbed preparation is necessary to ensure rapid germination and growth of the planted species. Applications of lime will help neutralize highly acidic topsoils. Similarly, fertilizers should be added for cover soils low in essential plant nutrients. Optimum soil application rates or lime and fertilizers should be determined from site-specific soil tests. Where required, lime should be worked to 6-inch depths into the soil by discing or harrowing (USEPA, 1976). For dense, impervious topsoils, loosening by tillage is recommended. Grading methods for loosening and roughening large areas of soils are discussed in Section 3.2.1.

Seeding should be performed as soon as possible after final grading and seedbed preparation (USEPA, 1976). The most common and efficient method of seeding large areas of graded slopes is with hydroseeders. Seed, fertilizer, mulch, and lime can be sprayed from hydroseeders onto steep outslopes and other areas of difficult access. Rear-mounted blowers can be attached to lime trucks to spread seed and fertilizers over such areas, also. Grass or grain drills may be used to apply seed on gently rolling or level, stone-free terrain. Hand planting, a time-consuming and costly project, may be required for trees and shrubs (USEPA, 1976).

Planting is often performed in stages so that complete revegetation may take several years. When planting large trees, it is important to know the depth and locations of protective caps and barriers so that they are not accidentally damaged.

Mulches are usually applied after seeding and fertilization, although certain mulch materials (e.g., wood fibers) may be applied as hydroseeder slurries mixed with seed, fertilizer, and lime (USEPA, 1976). Mulch application rates will vary depending on local climate, soil characteristics, and slope steepness.

Loose straw and hay mulches are the most common and most cost-effective temporary soil stabilizer/mulching materials available (USEPA, 1976). These mulches are best applied using a mulch blower, at rates from 1/2 to 4 tons per acre. Straw/hay mulches can be anchored to the soil by asphalt, chemical binders, or jute netting (USEPA, 1976).

Total 1985 cost estimates for revegetation range from $1259/acre to $8300/acre based on cost data presented in the cost compendium (USEPA, 1985). Cost for revegetation vary widely depending upon the site conditions. The lowest cost estimate was based on a hypothetical site which required hydroseeding (lime, fertilizer, field seed) only. The highest cost estimate was for a proposed restoration of a secondary growth, temperate, deciduous forest, requiring heavy lining to neutralize the highly acidic soils.

1985 Opertion and Maintenance costs for a revegetated area have been estimated to range from \$58/acre/year to \$1314/acre/year.

3.4.5 Operation, Maintenance, and Monitoring

Periodic reliming and fertilization may be necessary to maintain optimum yearly growth on seeded plots. Soils with poor buffering capacity may require frequent liming to achieve suitable pH levels; these are generally soils high in organic matter or clay content. Annual fertilization of nitrogen, phosphorus, or potassium deficient soils will also aid reclamation efforts. Fertilizer application rates will vary with the nutrient content and pH level of the seeded cover soil. Periodic mowing and the judicious use of selective herbicides will help control undesirable weed and brush species. Grass sodding and remulching or planting new shrubs and trees is recommended for sparsely covered, erosion-prone areas (Lutton, Regan, and Jones, 1979).

All vegetation should be monitored for sickness and death as these occurrences may be indicative of contamination in ground or surface waters.

3.4.6 Technology Selection/Evaluation

A well-designed and properly implemented revegetation plan--whether for long-term reclamation or short-term remedial action--will effectively reduce erosion and stabilize the surface of a covered disposal site, thereby improving the effectiveness and reliability of the cap. A multi-layered capping system and properly graded slopes, in combination with suitable vegetative cover, will eventually isolate buried wastes from surface hydrologic input.

Although vegetative cover requires frequent maintenance, it actually prevents more costly maintenance which would result from erosion by surface soils. Revegetation is also essential to the integrity and performance of dikes, waterways, and sedimentation basins.

3.4.7 Costs

Unit costs associated with revegetation of covered disposal sites are reported in Table 3-6. All costs are reported in 1985 dollars and include contractors overhead and profit.

3.5 Surface Water Diversion and Collection

This section discusses the various surface water diversion and collection methods in terms of their applications and limitations and design and

TABLE 3-6.
1985 UNIT COSTS ASSOCIATED WITH REVEGETATION OF COVERED DISPOSAL SITES

Description	Unit Costs[1]	Source
Hydraulic spreading (hydroseeding), lime, fertilizer, and seed	\$849.57/acre	2
Mulching, hay	\$309.09/acre	2
Loam topsoil	\$6.50/yd^3	2
Loam topsoil, remove and stockpile on-site using 200 hp dozer 6 ft deep		
200 ft haul	\$1.16/yd^3	3
500 ft haul	\$4.44/yd^3	3
Hauling loam on-site (2 miles)	\$3.84/yd^3	2
Spreading loam, 2-6 inches deep		
slopes	\$1.04 - \$3.12/yd^3	2
level areas	\$0.77 - \$2.27/yd^3	2
Plant bed preparation (unspecified), 18 inches deep, by machine	\$5.40/yd^2	3
Hydraulic seeding and fertilization of large areas with wood fiber mulch	\$0.41/yd^2	3
Mulch, hand spread 2 inches deep, wood chips	\$1.23/yd^2	3
Liming		
level areas	\$87.96/acre	2
slopes	\$570.82/acre	2

(continued)

TABLE 3-6. (continued)

Description		Unit Costs[1]	Source
Fertilizing (no insecticides)			
level areas		$448.01/acre	2
slopes		$641.33/acre	2
Seeding			
level areas		$656.78/acre	2
slopes		$850.11/acre	2
Jute mesh, stapled (erosion control)		$\$\ 0.85/\text{yd}^2$	3
Sodding, in East, 1 inch deep			
level areas		$\$2.85/\text{yd}^2$	3
slopes		$\$3.28/\text{yd}^2$	
Maintenance:			
Grass mowing			
slopes		$90.71/acre	2
level areas		$37.86/acre	2
Refertilization		$242.04/acre	2
Weeding/pruning shrubs		$1,943.87/acre	2
On-Site Planting			
Trees			
evergreens	30-36 inches	$112.19 ea.	2
	36-42 inches	$114.25 ea.	2
	42-48 inches	$160.31 ea.	2
	4-5 ft	$202.49 ea.	2
	5-6 ft	$258.20 ea.	2
Black Pines	2.5-3 ft	$27 ea.	3
Yews	2-2.5ft	$37 ea.	3
Junipers	4-5 ft	$49 ea.	3

(continued)

TABLE 3-6. (continued)

Description		Unit Costs[1]	Source
Shade trees (balled and burlapped)			
	6-8 ft	$76.65 ea.	2
	8-10 ft	$108.02 ea.	2
	1.5 - 2.5 inch diameter	$276.31 ea.	2
	2.5 - 4.0 inch diameter	$451.09 ea.	2
Birch	6-8 ft	$96 ea.	3
Oak	8-10 ft	$490 ea.	3
Shrubs (balled and burlapped)			
	2-3 ft	$55.57 ea.	2
	3-4 ft	$87.40 ea.	2
	4-5 ft	$106.20 ea.	2
Honeysuckle shrub	3-4 ft	$18.15 ea.	3

(continued)

---

[1] All costs include materials and installation (labor and equipment), unless otherwise indicated. Note different units (acre, square yard, cubic yard, each).

[2] McMahon, 1984.

[3] Godfrey, 1984.

construction considerations. Costs of these systems and factors affecting selection and evaluation are discussed in Sections 3.5.9 and 3.5.8, respectively.

3.5.1 Dikes and Berms

3.5.1.1 General Description

Dikes and berms are well-compacted earthen ridges or ledges constructed immediately upslope from or along the perimeter of disturbed areas (e.g., disposal sites). These structures are generally designed to provide short-term protection of critical areas by intercepting storm run-off and diverting the flow to natural or manmade drainage ways, to stabilized outlets, or to sediment traps. The two terms, dikes and berms, are generally used interchangeably; however, dikes may also have applications as flood containment levees (Section 3.5.7).

3.5.1.2 Applications/Limitations

Dikes and berms may be used to prevent excessive erosion of newly constructed slopes until more permanent drainage structures are installed or until the slope is stabilized with vegetation (USEPA, 1976). They are widely used to provide temporary isolation of wastes until they can be removed or effectively contained. They have particularly widespread use during excavation and removal operations where it is necessary to isolate drums or contaminated soils which have been temporarily staged on-site. The dikes not only prevent runoff, but can also prevent mixing of incompatible wastes. These temporary structures are designed to handle relatively small amounts of runoff; they are not recommended for drainage areas larger than 5 acres (Virginia SWCC, 1980). Diversion of storm run-off will decrease the amount of water available to infiltrate the soil cover, thereby reducing the amount of leachate production.

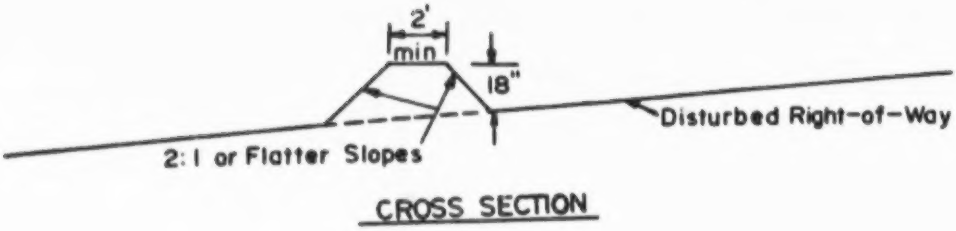
Dikes and berms are temporary structures which are usually used for no more than one year. Though construction costs are not usually high, the ultimate removal of the structures entails additional costs.

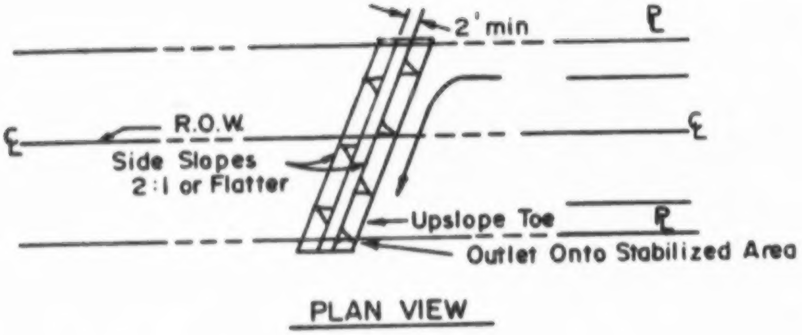
3.5.1.3 Design Considerations

Specific design and construction criteria for berms and dikes will depend upon desired site-specific functions of the structures. Run-off control dikes are generally classified into two groups:

- Interceptor dikes (Figure 3-7) which are built with 0 (zero) percent grade and are designed only to reduce slope length

FIGURE 3-7.
TEMPORARY INTERCEPTOR DIKE





Source: Virginia SWCC, 1974

- Diversion dikes (Figure 3-8) which are built with a grade sufficient to drain and are designed to intercept and divert surface flow as well as to reduce slope length.

Dikes and berms ideally are constructed of erosion-resistant, low permeability, clayey soils. Compacted sands and gravel, however, may be suitable for interceptor dikes and berms. The general design life of these structures is on the order of one year maximum; seeding and mulching or chemical stabilization of dikes and berms may extend their life expectancy. Stone stabilization with gravel or stone rip-rap immediately upslope of diversion dikes will also extend performance life.

A detailed design is not usually required before constructing interceptor and diversion dikes/berms. The following design criteria are commonly used:

- Drainage area: 5 acres maximum
- Top width: 2 feet minimum
- Height (compacted): 18 inch minimum
- Side slopes: 2:1 or flatter
- Grade: Dependent upon topography, but must have positive drainage to the outlet; for interceptor dikes/berms, 1.0 to 1.5 percent.
- Spacing (interceptor only):

Distance between dikes	150 ft	200 ft	300 ft
Maximum slope of area above dike	>10%	5-10%	<5%

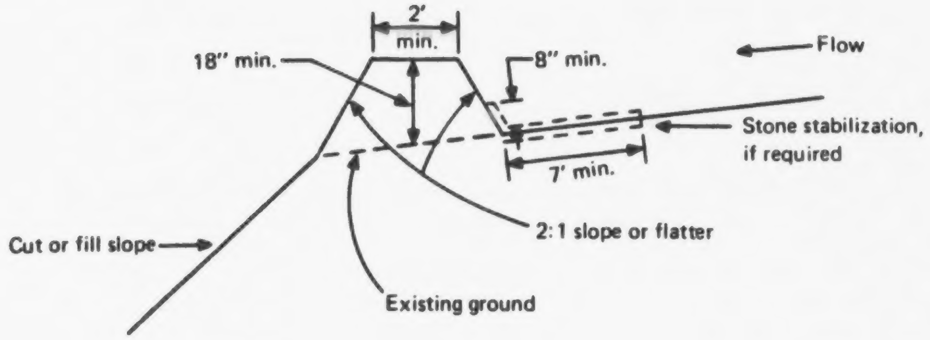
3.5.1.4 Construction/Implementation Considerations

Dikes and berms are constructed by well-established techniques, and very often the required excavation and grading equipment is already available at the disposal site. The required earth fill is often available on-site as well.

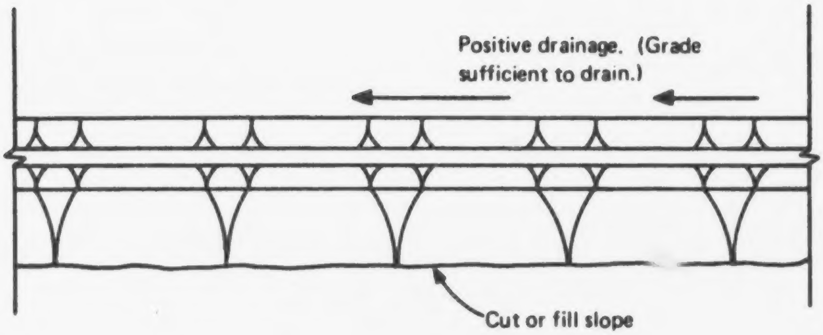
Structures of this type are often constructed in several lifts with each lift being tested for adequate compaction. The following considerations are essential for the construction of a proper dike:

- All earthen dikes should be machine compacted.
- All diversion dikes must have positive grade to an outlet.
- Diverted run-off should outlet directly onto stabilized areas, level spreader, grassed channel, or chute/downpipe.
- Periodic inspection and maintenance should be provided.

FIGURE 3-8.
TEMPORARY DIVERSION DIKE



Cross-section



Plan view

Source: USEPA, 1976

- Diversion dikes must be seeded and mulched immediately after construction.

3.5.1.5 Operation, Maintenance, and Monitoring

Adequate maintenance of dikes and berms is essential. Periodic inspections and maintenance are required to ensure structural integrity and prevent upslope deposition of sediments. When operating improperly, dikes and berms will actually act to increase seepage. It is generally not a good practice to utilize dikes or berms for a period of time exceeding one year.

3.5.1.6 Costs

The costs of installing and maintaining dikes and berms will vary with the site conditions, depending on: number and size of structures required, local availability of suitable soil and equipment, local climate and site hydrology (intensity and volume of storm run-off to be diverted), amount of maintenance required, design life of the structures, amount and type of stabilization required (seeding, mulching, chemical soil additives), and unforeseen construction difficulties.

Unit costs associated with dike/berm construction and maintenance are presented in Table 3-13 at the end of this chapter.

3.5.2 Channels and Waterways

3.5.2.1 General Description

Channels are excavated ditches that are generally wide and shallow with trapezoidal, triangular, or parabolic cross sections. Diversion channels are used primarily to intercept run-off or reduce slope length. They may or may not be stabilized. Channels stabilized with vegetation or stone rip-rap (waterways) are used to collect and transfer diverted water off site or to on-site storage or treatment (Ehrenfelder and Bass, 1983).

3.5.2.2 Applications/Limitations

Applications and limitations of channels and waterways differ depending upon their specific design. Table 3-7 summarizes the applications of specific types of channels and waterways with respect to drainage area and maximum permissible velocity. Earthen channels can be used on the perimeter of a disposal site to divert runoff from entering the area of waste disposal.

TABLE 3-7.
TYPICAL DESIGN CRITERIA FOR SWALES, CHANNELS, AND WATERWAYS

Technology	Drainage Area	Maximum Permissible Velocity (ft/sec)	Depth	Duration of Normal Use	Maximum Side Slopes	Stabilization
Earthen Channels	Less than 5 acres	2	1 ft minimum	Temporary	2:1	Grass or soil additives desirable if used for considerable time
CMP Channels	No size limitation	10	radius of pipe	Temporary	--	--
Sodded Waterways	No size limitation	5	0.4 to 3 ft (parabolic) 0.5 to 6 ft (trapezoidal)	Permanent	2:1	Sod, stone bottom for base flow
Stone Waterways	No size limitation	6	1 ft minimum	Permanent	2:1	--

Source: SCS Engineers, 1980

Figure 3-9 is a cross section of a typical earthen channel. They are usually temporary, since they are inadequately stabilized to resist erosion.

A diversion is a modification of an earthen channel. A diversion is a shallow channel excavated along the contour of graded slopes and having a supporting earthen ridge (dike or berm) constructed along the downhill edge of the channel. Essentially, a diversion is a combination of a ditch and a dike (USEPA, 1976). Figure 3-10 illustrates a diversion.

Swales placed along the perimeter of the site are used to keep off-site runoff from entering the site and to carry surface runoff from landfills. They are distinguished from earthen channels by side slopes which are less steep and have vegetative cover for erosion control.

Half-round pipe can be used in a manner similar to earthen channels. It can be constructed by cutting corrugated metal pipe (CMP) or can be purchased in an asbestos impregnated asphalt called sectional slope drains. Channels are constructed from half-round pipe which is placed below grade. Half-round pipe channels are easier to install and have lower maintenance costs than

FIGURE 3-9.
TYPICAL DRAINAGE DITCH AT BASE OF DISPOSAL SITE

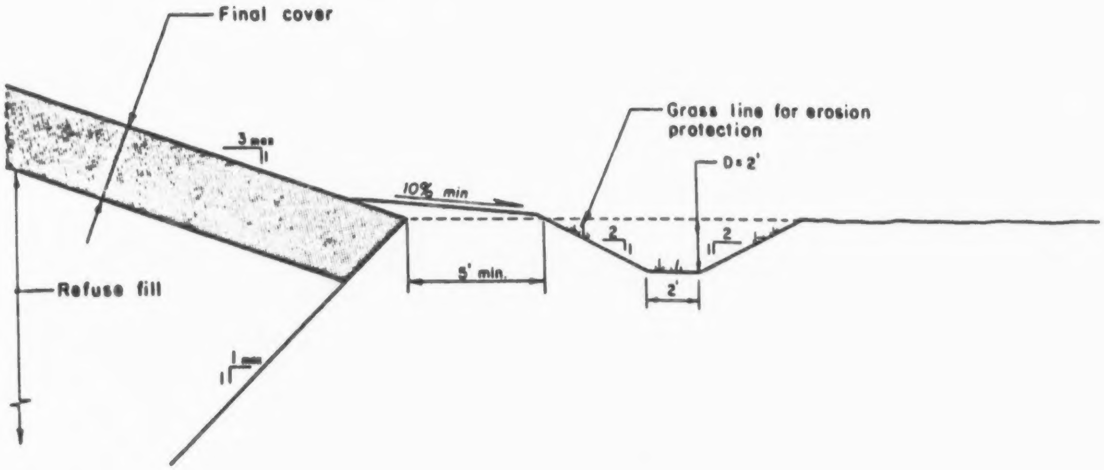
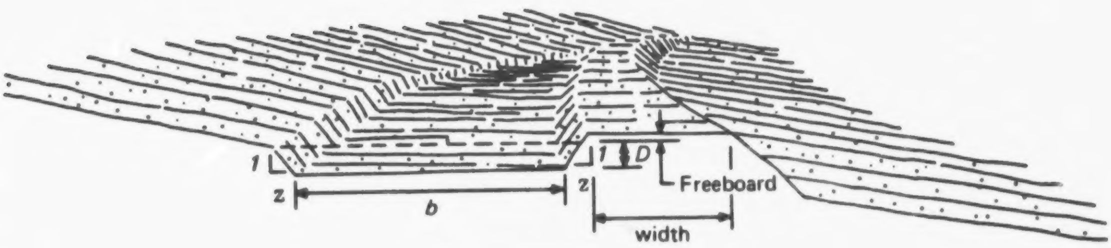
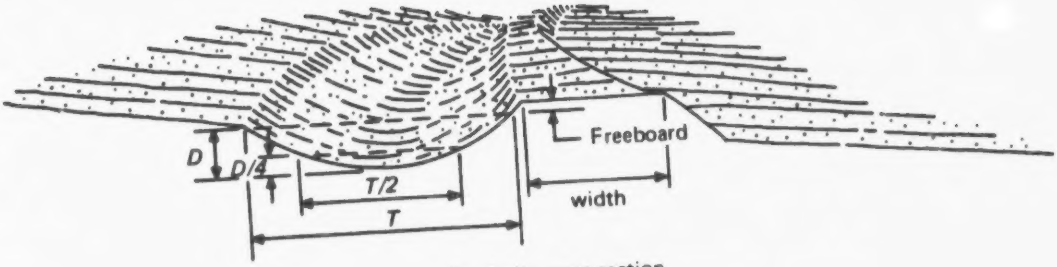


FIGURE 3-10.
GENERAL DESIGN FEATURES OF DIVERSIONS



Trapezoidal cross-section



Parabolic cross-section

Source: USEPA, 1976

earthen channels. In addition, infiltration into the site is minimized. Like earthen channels, half-round channels may be constructed on the perimeter of a waste site and moved as needed to protect other portions of the waste site. They can also be used successfully to carry storm water run-off over a filled area where it is not practical to carry the run-off around the fill.

Sodded waterways (Figure 3-11) are used primarily in situations where the flow quantities and velocities are significant and a full growth of grass is required to prevent erosion. Typically, five acres is often considered to be a large enough area to warrant sod. The maximum allowable velocities for sodded waterways are dependent on the type of grass.

Where the tributary drainage area is relatively small and the quantity of run-off and velocity of flow is low, grass-seeded waterways can be effectively used.

Stone waterways provide added protection against erosion. If the waterway is intended to carry large quantities of run-off and/or the flow velocities are excessive, the stone may be grouted in place. A stone channel is a permanent means of diverting off-site drainage around the disposal site and carrying surface run-off from landfills. Figure 3-12 illustrates a grassed waterway with a stone center.

3.5.2.3 Design Considerations

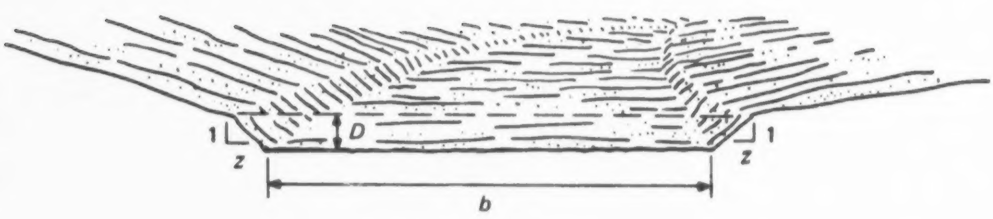
Channels, swales, and waterways are generally of V-shaped, trapezoidal, or parabolic cross-section design. The specific design will be dependent on local drainage patterns, soil permeability, annual precipitation, area land use, and other pertinent characteristics of the contributing watershed. Table 3-7 gives some general guidelines for depth and maximum side slopes for various channel and waterway designs. Figure 3-13 shows a standard design for an earthen channel.

Channels and waterways should be designed to accommodate flows resulting from rainfall events (storms) of 10- or 25-year frequency. More importantly, they should be designed and constructed to intercept and convey such flows at nonerosive velocities.

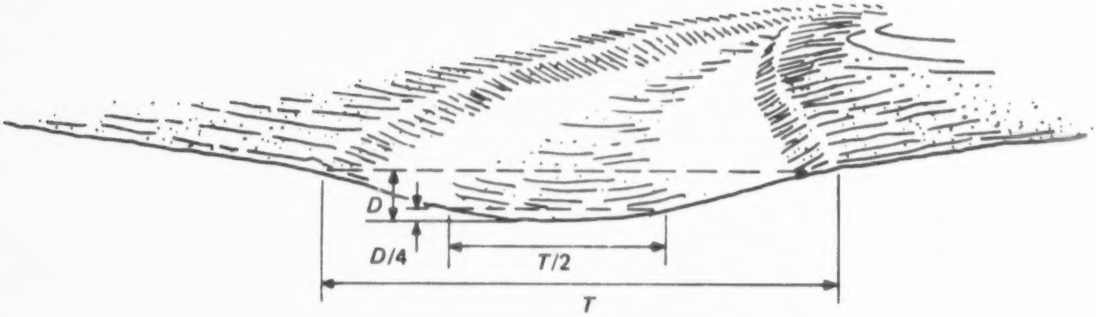
Figure 3-14 depicts the effect of drainage channel shape on relative velocity of conveyed flows. In general, the wider and shallower the channel cross-section, the less the velocity of contained flow, and, therefore, the less the potential for erosion of drainageway side slopes. Where local conditions dictate the necessity of building narrower and deeper channels, or where slopes are steep and flow velocities are excessive, the channel will require stabilization through either seeding and mulching or the use of stone rip-rap to line channel bottoms and break up flow.

Design of channel and waterways is generally based on the Manning formula for steady uniform flow in open channel.

FIGURE 3-11.
GRASSED WATERWAYS



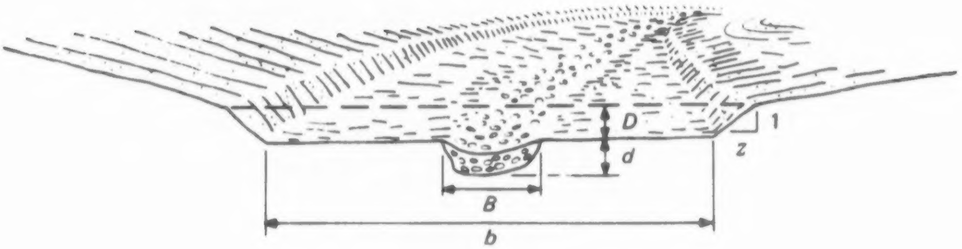
Trapezoidal cross-section



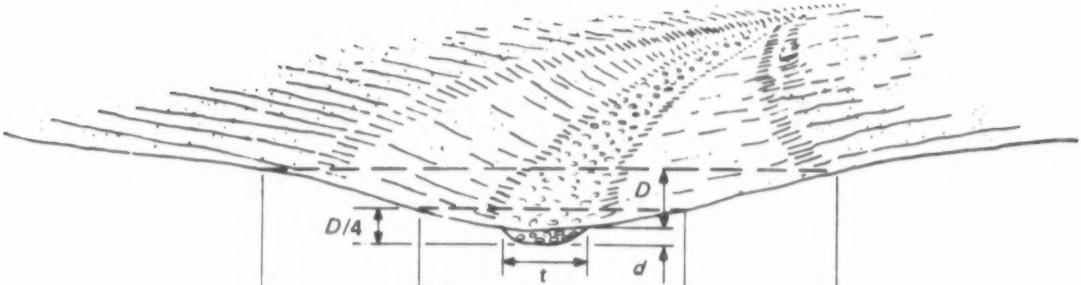
Parabolic cross-section

Source: USEPA, 1976

FIGURE 3-12.
GRASSED WATERWAYS WITH STONE CENTERS



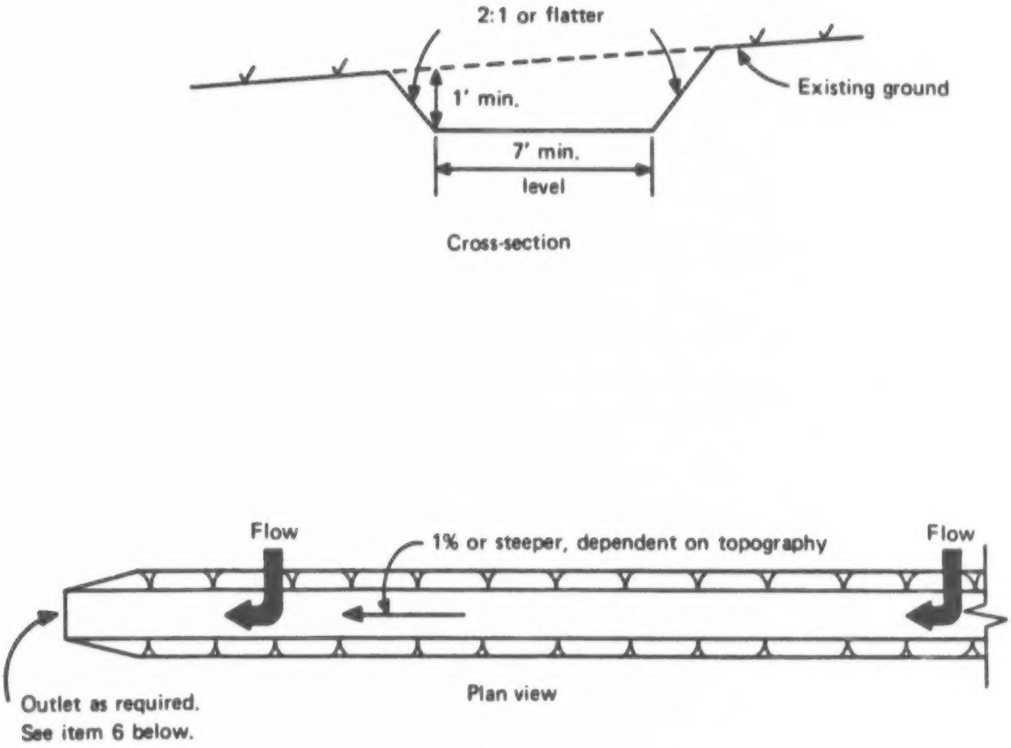
Trapezoidal cross-section



Parabolic cross-section

Source: USEPA, 1976

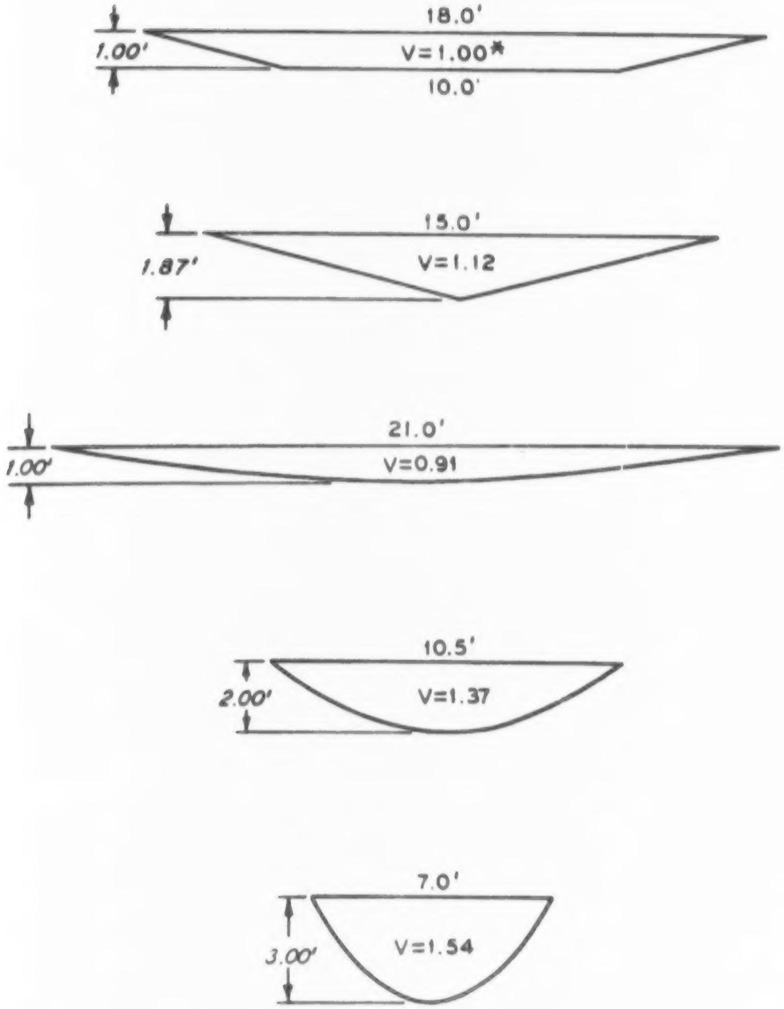
FIGURE 3-13.
STANDARD DESIGN FOR DRAINAGE DITCHES



Source: USEPA, 1976

Figure 3-13. Standard Design for Drainage Ditches

FIGURE 3-14.
EFFECT OF DRAINAGE DITCH ON VELOCITY



*V=RELATIVE VELOCITY

Source: Lutton, Regan, and Bass, 1983

The Manning formula is considered when designing for steady uniform flow in open channels (Ehrenfelder and Bass, 1983):

$$V = \frac{Cm}{n} R^{2/3} S^{1/2}$$

and for an open channel of cross-sectional area, A:

$$Q = \frac{Cm}{n} A R^{2/3} S^{1/2}$$

where:

Q = design capacity (m^3/sec)

R = hydraulic radius (area divided by wetted perimeter) (m)

A = cross-sectional area of the channel (m^2)

S = channel slope (m/m)

C_m = dimensionless constant (1.0 for metric units)

n = Manning roughness factor (sec/$m^{1/3}$). Values of n for various materials are given in Table 3-8.

Permissible flow velocities for channels lined with vegetation are given in Table 3-9.

Channel spacing (when used for interception or slope length reduction) depends on the slope of the area above the channel (USEPA, 1976):

Slope	Distance Between Channels
>10%	30m (100 ft)
5-10%	60m (200 ft)
<5%	90m (300 ft)

3.5.2.4 Construction/Implementation Considerations

The construction of channels and waterways does not generally present any complications as simple and well-established construction techniques are generally used. Where there is extensive gullying, considerable earthwork will be required. Compaction of fills should be performed as needed to prevent unequal settling. All trees, bushes, stumps, and obstructions should be cleared to prevent improper functioning of the channel. If local fill is used, it must meet design standards and be free of contamination.

TABLE 3-8.
VALUE OF MANNING'S n FOR VARIOUS CHANNEL SURFACE MATERIALS

Material	Suggested n[a,b]
Planed Wood	0.012
Unplaned wood	0.013
Finished concrete	0.012
Unfinished concrete	0.014
Cast iron	0.015
Brick	0.016
Riveted steel	0.018
Corrugated metal	0.022
Rubble	0.025
Earth	0.025
Earth with stones or weeds	0.035
Gravel	0.029
Vegetation	.04[c]

[a] The Manning forumla is an empirical formula. The dimensions of Cm and n are therefore somewhat arbitrary. In metric units, CM = 1.0 and n is in (s)/($M^{1/3}$). In English units, Cm = 1.486 ($ft^{1/2}$)/(s) and n is in $ft^{1/6}$. The numerical values for n, however, do not change.

[b] In situations with R>3m, roughness factor should be increased by 10 to 15%.

[c] From TRD 8.

Source: Streeter and Wylie, 1975

TABLE 3-9.
PERMISSIBLE DESIGN VELOCITIES FOR STABILIZED CHANNELS AND WATERWAYS

Vegetative Cover	Channel grade (%)	Maximum design velocity (ft/sec)
Bermuda grass	0-5	6
	5-10	5
	>10	4
Reed canary grass, Tall fescue, or Kentucky bluegrass	0-5	5
	5-10	4
	>10	3
Grass-legume mix	0-5	4
	5-10	3
Red fescue, or Redtop (sericea lespedeza)	0-5	2.5
Annuals; Small grain (rye, oats, barley); Ryegrass	0-5	2.5

Source: USEPA, 1976

3.5.2.5 Operation, Maintenance, and Monitoring

Failure of channels and waterways may result from insufficient capacity, excessive velocity, or inadequate vegetative cover. The first of these two are largely a matter of inadequate design. Grassed waterways must be periodically mowed to prevent excessive retardation of flow and subsequent ponding of water. Also, periodic resodding, remulching, and fertilizing may be required to maintain vegetated channels. Sediment accumulation often results in failure of channels and waterways. Control of vegetation to prevent matted growth and high allowable design velocities will reduce sediment accumulation. Stone channels have the advantage of requiring minimal maintenance.

It may be necessary to install temporary straw-bale check dams, staked down at 50- to 100-foot intervals, across channels and waterways in order to prevent gully erosion and to allow vegetative establishment (Tourbier and Westmacott, 1974). The installation and ultimate removal of these check dams will add to the costs associated with diversions and waterways.

3.5.2.6 Costs

As with all surface water diversion structures, costs for ditches, diversions, and waterways will be highly variable because of site-specific differences in fill materials and equipment availability, design capacity and life expectancy of the structure, etc. Unit costs associated with the construction and maintenance of these structures are presented at the end of this chapter in Table 3-13.

3.5.3 Terraces and Benches

3.5.3.1 General Description

Terraces and benches are embankments constructed along the contour of very long or very steep slopes to intercept and divert flow and to control erosion by reducing slope length. These structures are classified as bench terraces or drainage benches. Bench terraces are used primarily to reduce land slope while drainage benches on broadbased terraces act to remove or retain water on sloping land.

3.5.3.2 Applications/Limitations

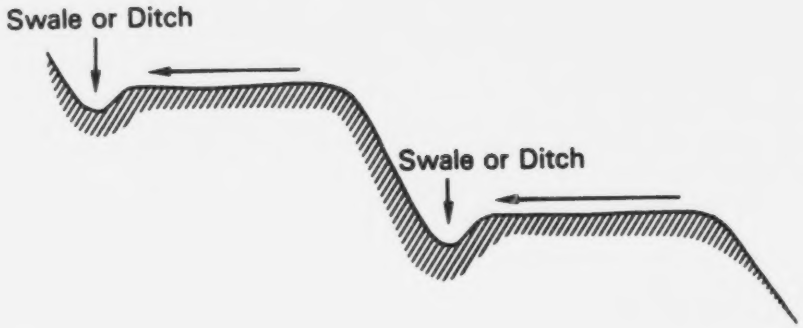
Benches and terraces may be used to break up steeply graded slopes of covered disposal sites into less erodible segments. Upslope of disposal sites, they act to slow and divert storm run-off around the site, thereby minimizing erosion. Downslope of landfill areas, they act to intercept and divert sediment-laden run-off to traps or basins. Hence, they may function to hydrologically isolate sites where remedial actions have not yet been completed, to control erosion of cover materials on sites which have been capped, or to collect contaminated sediments eroded from disposal areas. For disposal sites undergoing final grading (after capping and prior to revegetation), construction of benches or terraces may be included as part of the integrated site closure plan.

3.5.3.3 Design Considerations

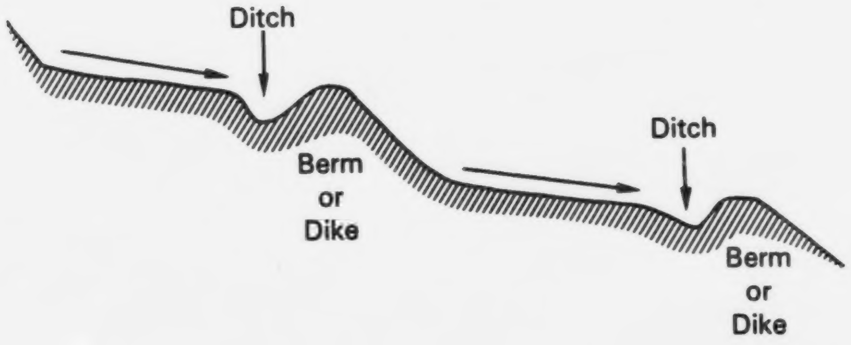
Although benches and terraces are slope reduction devices, they are generally constructed with reverse fall or natural fall (see Figure 3-15). Figure 3-16 presents the design for a typical drainage bench located on the slope of a covered landfill. This particular bench is designed with a natural fall. It is intended for long-term erosion protection as the associated V-shaped channel is asphalt concrete-lined. Diversions and ditches included in bench/terrace construction may be seeded and mulched, sodded, stabilized

FIGURE 3-15.
BENCH TERRACES

With Reverse Fall



With Natural Fall



Adapted from: Tourbier and Westmacott, 1974 with rip-rap or soil additives, or stabilized by any combination of these methods. Lining the channels with concrete or grouted rip-rap is a more costly alternative.

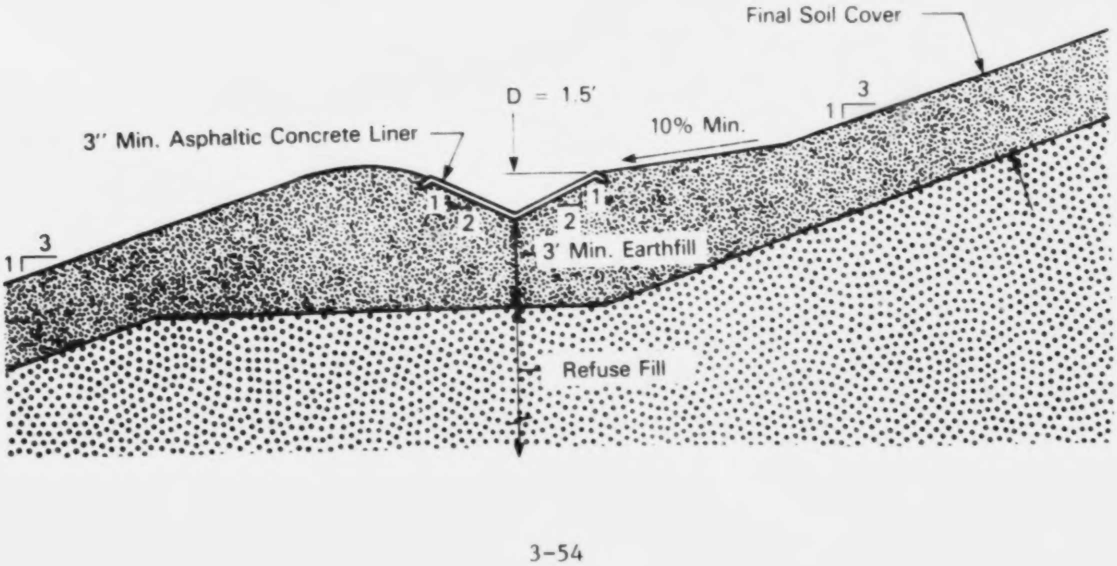
The design of a terrace system involves the proper spacing and location of terraces and the design of a channel with adequate capacity.

The spacing between benches and terraces will depend on slope steepness, soil type, and slope length. In general, the longer and steeper the slope and the more erodible the cover soil, the less the distance between drainage benches should be. This will maximize the erosion reduction afforded by constructed benches. For slopes greater than 10 percent in steepness, the maximum distance between drainage benches should be approximately 100 feet, i.e., a bench every 10 feet of rise in elevation (USEPA, 1976). When the slope is greater than 20 percent, it has been recommended that benches be placed every 20 feet of rise in elevation (USEPA, 1976). An exact solution to terrace spacing can be found in Schwab et al. (1981).

Benches should be of sufficient width and height to withstand a 24-hour, 25-year storm (USEPA, 1976).

The terrace channel acts as a temporary storage reservoir subjected to unequal rates of inflow and outflow. For graded terraces the Manning velocity equation given in Section 3.5.2 is suitable for design. A roughness coefficient of 0.04 is usually selected for tilled soil and design is based on this condition because overtopping would cause the worst damage. The maximum

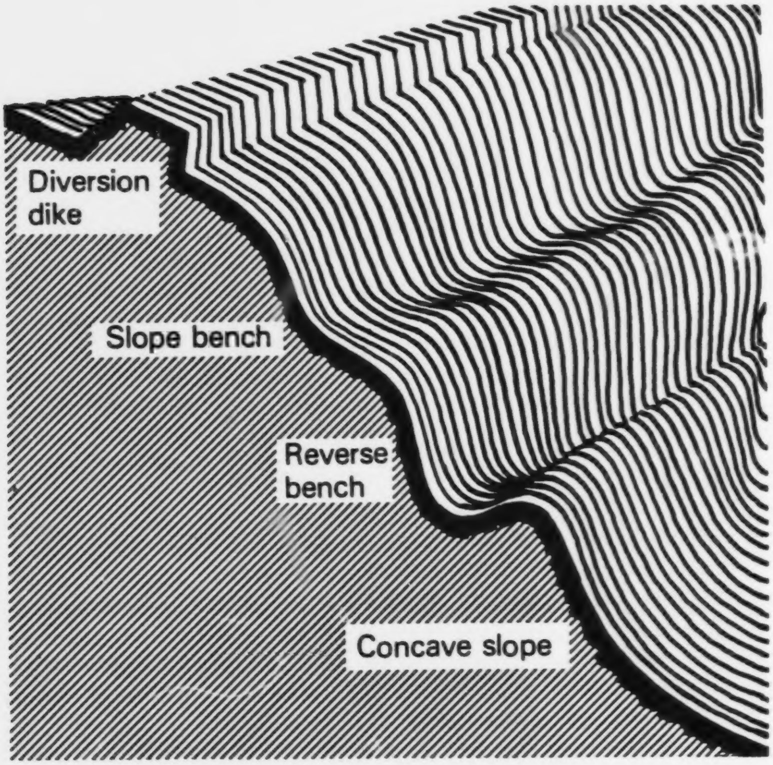
FIGURE 3-16.
TYPICAL DRAINAGE BENCH



design velocity will vary with the erosiveness of the soil but should rarely exceed 0.6 m/sec for soil devoid of vegetation. The channel depth should permit a freeboard of about 20 percent of the total depth after allowing for settlement of the ridge (Schwab et al., 1981).

Bench terraces do not necessarily have to be designed with diversions or ditches to intercept flow. Reverse benches and slope benches (Figure 3-17) may be constructed during final site grading on well-stabilized slopes (e.g., vegetated) to enhance erosion control by reducing slope length and steepness. At sites where an effective cap (e.g., clay or synthetic liner) has been constructed, or for sites located in arid regions, these nondrainage benches

FIGURE 3-17.
SLOPE REDUCTION MEASURES



Source: USEPA, 1976

will function to slow sheet run-off and allow greater infiltration rates, which will aid in the estabishment of a suitable vegetative cover. For most disposal sites in wet climates, however, where leachate generation and cover erosion are major problems, benches and terraces should be designed in association with drainage channels that intercept and transport heavy, concentrated surface flows safely off-site.

3.5.3.4 Construction/Implementation Considerations

Benches and terraces are constructed with a variety of commonly used excavation equipment including bulldozers, scrapers, and graders. As with other earthen erosion control structures, they are constructed using well-established techniques and with local fill material. Benches and terraces must be sufficiently compacted and stabilized with appropriate cover (grasses, mulches, sod) to accommodate local topography.

3.5.3.5 Operation, Maintenance, and Monitoring

Benches and terraces should be inspected frequently during the first year of operation and during or after major storms to ensure proper functioning and structural integrity. Terrace ridges may settle up to 25 percent depending upon construction techniques. If bench slopes become badly eroded or if their surfaces become susceptible to ponding from differential settling then regrading and sodding may be necessary.

If improperly designed or constructed, bench terraces will not perform efficiently and may entail excessive maintenance and repair costs. It is important that these structures be stabilized with vegetation as soon as possible after grading and compaction, or they may become badly eroded and require future resodding or chemical stabilization. Benches and terraces also require periodic inspections, especially after major rainfall events.

3.5.3.6 Costs

The costs of bench and terrace construction will depend on the amount of fill required, the local availability of fill materials and grading eqiupment, the size and type of diversion channels to be installed, and the local costs of seeding, mulching, and other stabilizing materials. The frequency and extent of required maintenance will add to these costs.

Unit costs for these construction and maintenance activities are presented in Table 3-13 at the end of this chapter.

3.5.4 Chutes and Downpipes

3.5.4.1 General Description

Chutes and downpipes are structures used to carry concentrated flows of surface run-off from one level to a lower level without erosive damage. They generally extend downslope from earthen embankments (dikes or berms) and convey water to stabilized outlets located at the base of terraced slopes.

Chutes (or flumes) are open channels, normally lined with bituminous concrete, portland cement concrete, grouted rip-rap, or similar non-erodible material.

Downpipes (downdrains; pipe slope drains) are temporary structures constructed of rigid piping (such as corrugated metal) or flexible tubing of heavy-duty fabric. They are installed with standard prefabricated entrance sections and are designed to handle flow from drainage areas of 5 acres or less.

3.5.4.2 Applications/Limitations

Chutes and downpipes often represent key elements in combined surface control systems. They are especially effective in preventing erosion on long, steep slopes, and can be used to channel storm run-off to sediment traps, drainage basins, or stabilized waterways for off-site transport. However, they provide only temporary erosion control while slopes are stabilized with vegetative growth. Chutes are limited to heads of about 18 feet or less (Schwab et al., 1981). Also downpipes are limited to drainage areas five acres in size.

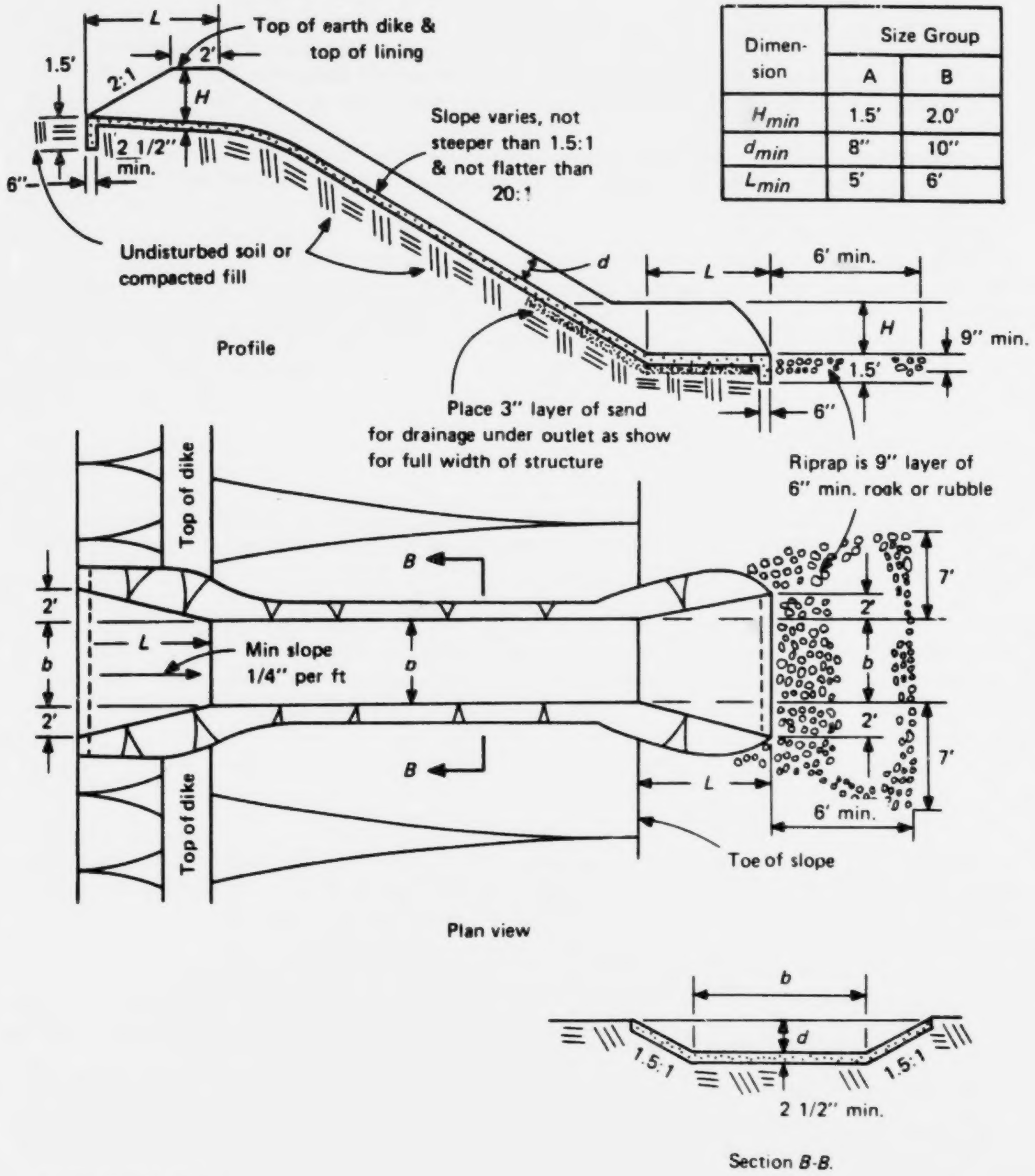
3.5.4.3 Design Considerations

Chutes and downpipes are temporary structures that do not require formal design. General design criteria are presented in Figures 3-18 (paved chute), 3-19 (rigid downpipe), and 3-20 (flexible downpipes).

Chutes are designed to handle flows based on two basic size groups. Paved chutes of size group A have the following three qualifications:

- Height (H) of dike at entrance = 1.5 feet minimum
- Depth (D) of chute down the slope = 8 inches minimum
- Length (L) of inlet/outlet sections = 5 feet minimum

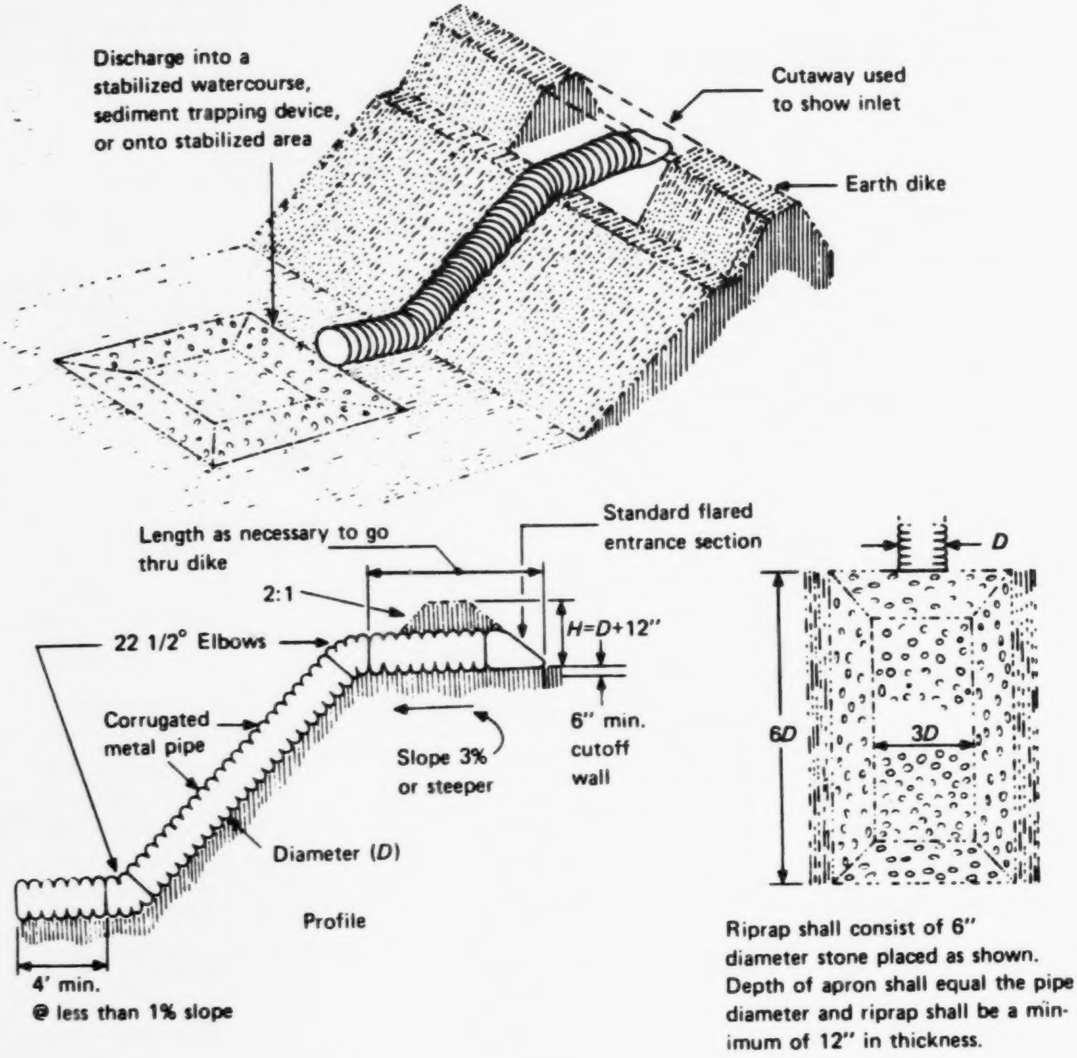
FIGURE 3-18.
PAVED CHUTE (OR FLUME)


L
1.5'
2'
Top of earth dike & top of lining
2:1
H
2 1/2" min.
6"
Slope varies, not steeper than 1.5:1 & not flatter than 20:1
Undisturbed soil or compacted fill
d
L
6' min.
H
9" min.
1.5'
6"
Profile
Place 3" layer of sand for drainage under outlet as show for full width of structure
Riprap is 9" layer of 6" min. rock or rubble
Top of dike
B
2'
L
Min slope 1/4" per ft
b
b
2'
B
7'
2'
b
2'
7'
L
6' min.
Toe of slope
Top of dike
Plan view
b
d
1.5:1
1.5:1
2 1/2" min.
Section B-B.

Dimension	Size Group	
	A	B
H_{min}	1.5'	2.0'
d_{min}	8"	10"
L_{min}	5'	6'

Source: USEPA, 1976

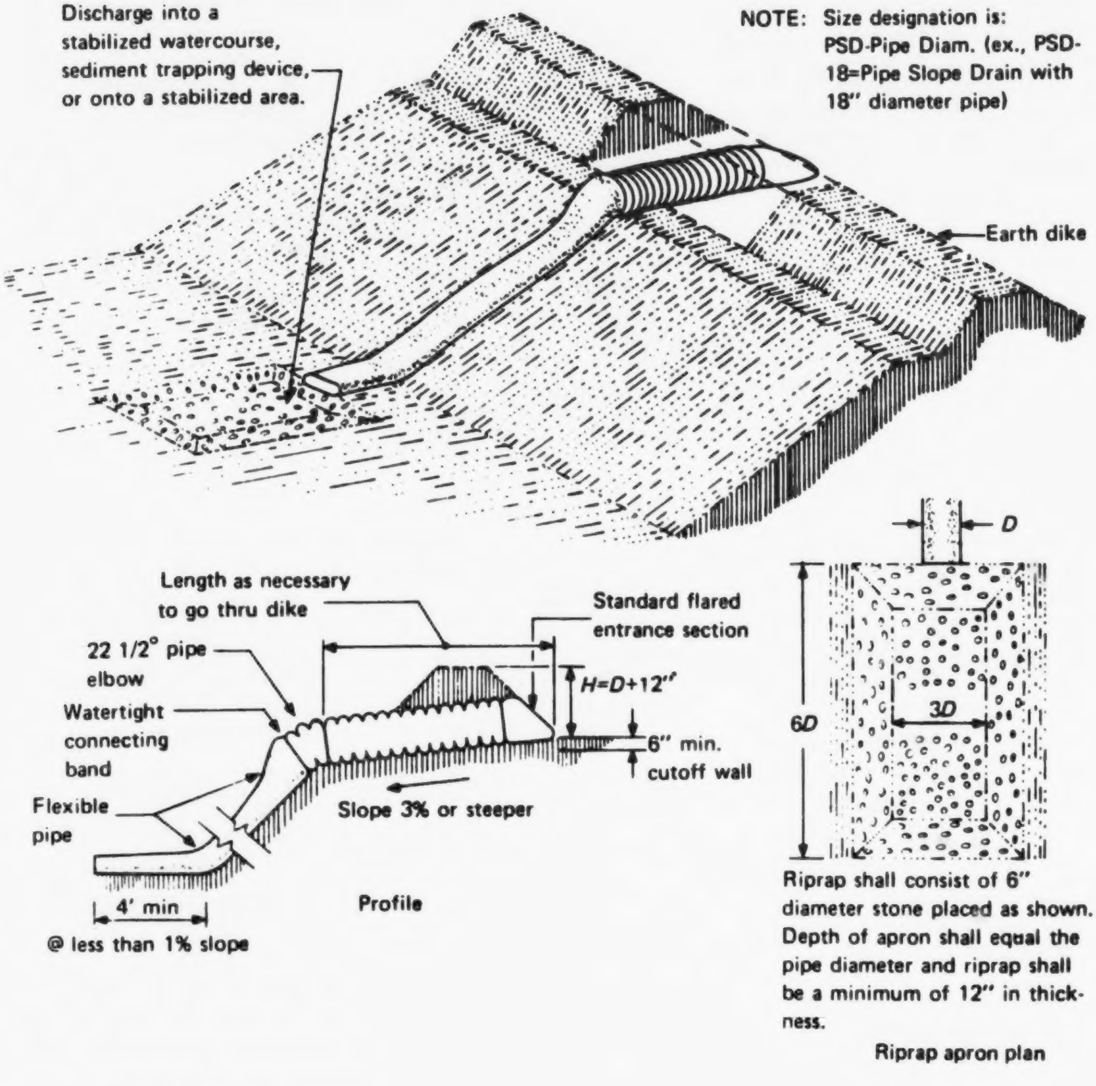
FIGURE 3-19.
RIGID DOWNPIPE


Discharge into a stabilized watercourse, sediment trapping device, or onto stabilized area
Cutaway used to show inlet
Earth dike
Length as necessary to go thru dike
Standard flared entrance section
2:1
22 1/2° Elbows
H=D+12"
Corrugated metal pipe
Slope 3% or steeper
6" min. cutoff wall
Diameter (D)
Profile
4' min.
@ less than 1% slope
D
6D
3D
Riprap apron plan

Riprap shall consist of 6" diameter stone placed as shown. Depth of apron shall equal the pipe diameter and riprap shall be a minimum of 12" in thickness.

Source: USEPA, 1976

FIGURE 3-20.
FLEXIBLE DOWNPIPE


Discharge into a stabilized watercourse, sediment trapping device, or onto a stabilized area.
Earth dike
Length as necessary to go thru dike
Standard flared entrance section
22 1/2° pipe elbow
Watertight connecting band
H=D+12"
6" min. cutoff wall
Flexible pipe
Slope 3% or steeper
4' min
@ less than 1% slope
D
6D
3D

NOTE: Size designation is: PSD-Pipe Diam. (ex., PSD-18=Pipe Slope Drain with 18" diameter pipe)

Profile

Riprap shall consist of 6" diameter stone placed as shown. Depth of apron shall equal the pipe diameter and riprap shall be a minimum of 12" in thickness.

Riprap apron plan

Source: USEPA, 1976

Similarly, chutes of size group B meet the following criteria:

- H = 2 feet minimum
- D = 10 inches minimum
- L = 6 feet minimum.

Table 3-10 below presents the bottom width and maximum drainage area for designed chutes of the two size groups.

TABLE 3-10.
BOTTOM WIDTHS AND MAXIMUM DRAINAGE AREAS FOR TEMPORARY CHUTES

Size group	Bottom width, D (ft)	Maximum drainage area (acres)	Size group	Bottom width, D (ft)	Maximum drainage area (acres)
A-2	2	5	B-4	4	14
A-4	4	8	B-6	6	20
A-6	6	11	B-8	8	25
A-8	8	14	B-10	10	31
A-10	10	18	B-12	12	36

[1] Source: USEPA, 1976.

If 75 percent or more of the drainage area has good vegetative cover (established grasses and/or shrubs) throughout the design life of the chute, the drainage areas listed in Table 3-9 may be increased by 50 percent. If 75 percent or more of the drained area has a mulch cover throughout the structure's life, the areas may be increased by 25 percent (USEPA, 1976).

For downpipes (Figures 3-19 and 3-20), the maximum drainage area is determined from the diameter of the piping, as follows (USEPA, 1976):

Pipe/Tubing Diameter, D (Inches)	Maximum Drainage Area (Acres)
12	0.5
18	1.5
21	2.5
24	3.5
30	5.0

3.5.4.4 Construction/Implementation Considerations

Construction methods for chutes and downpipes are inexpensive and quick. No special materials or equipment are required. Temporary downpipes may be constructed by joining half-round sections of bituminous fiber or concrete pipe. These structures may be quickly constructed for emergency situations when downslope ditches or waterways overflow during severe storms and threaten to erode the base of disposal fill areas.

General construction criteria for both rigid and flexible downdrains include the following:

- The inlet pipe shall have a slope of 3 percent or greater.
- For the rigid downpipe, corrugated metal pipe with watertight connecting bands shall be used.
- For the flexible downpipe, the inlet pipe shall be corrugated metal; the flexible tubing shall be the same diameter as the inlet pipe, securely fastened to the inlet with metal strapping or watertight connecting collars.
- A rip-rap apron shall be provided at the outlet; this shall consist of 6-inch diameter stone placed as shown in the figures.
- The soil around and under the inlet pipe and entrance sections shall be hand-tamped in 4-inch lifts to the top of the earth dike.

Paved chute construction considerations include the following (USEPA, 1976):

- The structure shall be placed on undisturbed soil or well compacted fill.
- The lining shall be placed by beginning at the lower end and proceeding upslope; the lining shall be well compacted, free of voids, and reasonably smooth.
- The cut-off walls at the entrance and at the end of the asphalted discharge aprons shall be continuous with the lining.
- An energy dissipator (rip-rap bed) shall be used to prevent erosion at the outlet.

3.5.4.5 Operation, Maintenance, and Monitoring

Periodic inspections and maintenance are necessary to ensure proper performance of chutes and downpipes. Downpipes should be inspected after each storm. These structures are intended for temporary use and should be removed when no longer needed.

3.5.4.6 Costs

Costs of chute and downpipe construction and maintenance will vary, depending on the size (length, width, depth) of the structure and the type of liner and pipe material used (corrugated metal, flexible drainpipe, bituminous fiber, PVC). Unit costs associated with these temporary structures are presented at the end of the chapter in Table 3-13.

3.5.5 Seepage Basins and Ditches

3.5.5.1 General Description

Seepage or recharge basins and ditches are used to discharge water collected from surface water diversions, groundwater pumping or leachate treatment to groundwater. They may also be used in in-situ treatment to force treatment reagents into the subsurface.

3.5.5.2 Applications/Limitations

Seepage basins and ditches are most effective in highly permeable soils so that recharge can be performed. They are not applicable at sites where collected run-off or groundwater is contaminated. Many basins and ditches are used in areas with shallow groundwater tables. Very deep basins or trenches can be hazardous.

Seepage ditches distribute water over a larger area than achievable with basins. They can be used for all soil where permeability exceeds about 0.9 inches per day.

3.5.5.3 Design Considerations

The major factors to consider in the design of seepage basins and ditches include (Healey and Laak, 1974):

- Amount of waste to be discharged
- Permeability of the surrounding soil
- Highest elevation of the water table
- Depth to impermeable stratum.

There is considerable flexibility in the design of seepage basins and ditches. Figures 3-21 through 3-24 illustrate possible design variations.

A seepage basin (Figure 3-21) typically consists of the actual basin, a sediment trap, a by-pass for excess flow, and an emergency overflow. A considerable amount of recharge occurs through the sidewalls of the basin, and it is preferable that these be constructed of pervious material. Gabions are frequently used to make sidewalls. An alternative design for a seepage basin is shown in Figure 3-22. This is not designed for an intensive recharge as the previously discussed system, and is usually used where the aquifer is shallow.

Seepage ditches illustrated in Figures 3-23 and 3-24 are generally used in parallel. Run-off is disposed of by a system of drains set in ditches of gravel. Depth and spacing of drains depends on soil permeability. A minimum depth of 48 inches is generally recommended, and ditches are rarely less than 10 feet apart. The ditches are backfilled with gravel, on which the distribution line is laid. Sediment is removed prior to discharging run-off into the seepage ditches by use of a sediment trap and distribution box. The efficiency of the seepage area can be increased by interconnecting two trenches by a continuous 12-inch gravel bed, as shown in Figure 3-24 (Tourbier and Westmacott, 1974).

3.5.5.4 Construction/Implementation Considerations

Seepage basins and ditches are constructed with well-established techniques and procedures. Much of the equipment and material necessary will be found on-site. Such things as piping and gravel may need to be specially ordered.

Dense turf on the side slopes of these basins will prevent erosion and sloughing and will also allow a high infiltration rate. Prevention of scour by the inlet is an important consideration since it can significantly reduce maintenance requirements. This can be accomplished by a "hydraulic jump" or an impact stilling basin before water flows into the recharge basin. Percolation can be improved by construction of gravel-filled trenches along the basin floor, as shown in Figure 3-21 (Toubier and Westmacott, 1974).

3.5.5.5 Operation, Maintenance, and Monitoring

Seepage systems can perform reliably only if they are well-maintained. Seepage basins and ditches are extremely susceptible to clogging by solids and biological growth and need to be periodically monitored and cleaned if necessary.

FIGURE 3-21.
SEEPAGE BASIN; LARGE VOLUME, DEEP DEPTH TO GROUNDWATER

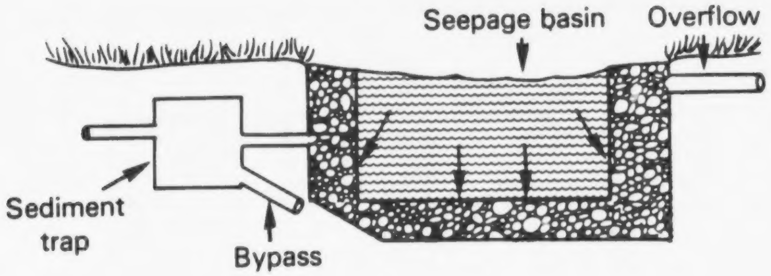
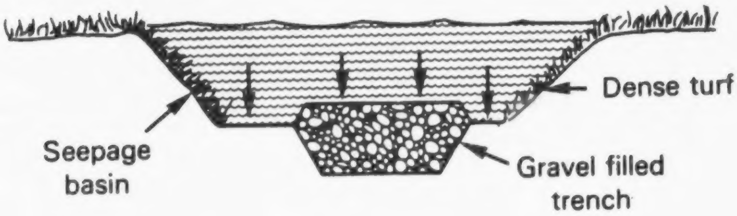


FIGURE 3-22.
SEEPAGE BASIN; SHALLOW DEPTH TO GROUNDWATER



Source: Tourbier and Westmacott, 1974

FIGURE 3-23.
SEEPAGE DITCH

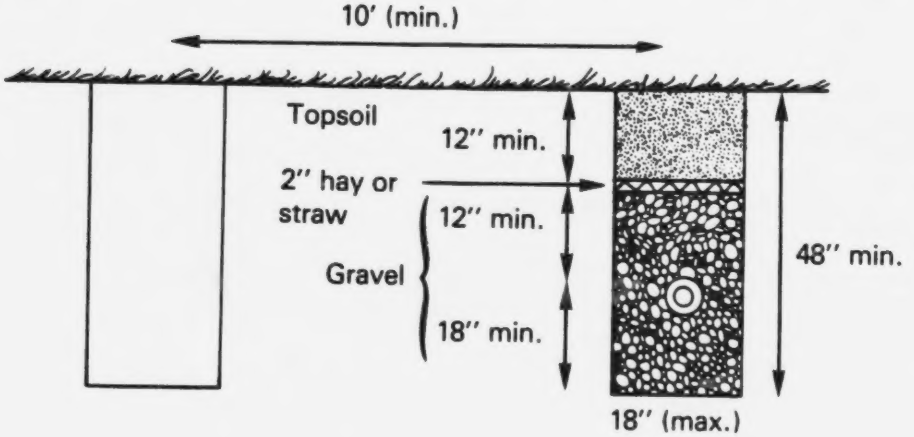
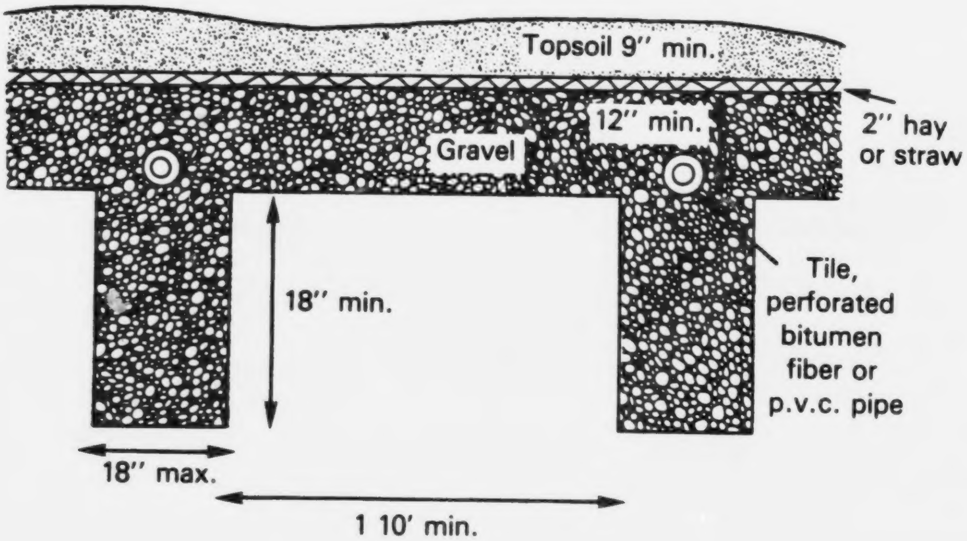


FIGURE 3-24.
SEEPAGE DITCH WITH INCREASED SEEPAGE EFFICIENCY



Source: Tourbier and Westmacott, 1974

3.5.5.6 Costs

The costs of seepage structures are highly variable depending on type, depth, length, necessary materials, and construction technique. Unit costs for excavated structures and associated material are tabulated in Table 3-13.

3.5.6 Sedimentation Basins/Ponds

3.5.6.1 General Description

Sedimentation basins are used to control suspended solids entrained in surface flows. A sedimentation basin is constructed by placing an earthen dam across a waterway or natural depression, or by excavation, or by a combination of both. The purpose of installing a sedimentation basin is to impede surface run-off carrying solids, thus allowing sufficient time for the particulate matter to settle.

3.5.6.2 Applications/Limitations

Sedimentation basins are usually the final step in control of diverted, uncontaminated, surface run-off, prior to discharge. They are especially useful in areas where there exists a high silt or sand content in the surface run-off. They are an essential part of any good surface flow control system.

3.5.6.3 Design Considerations

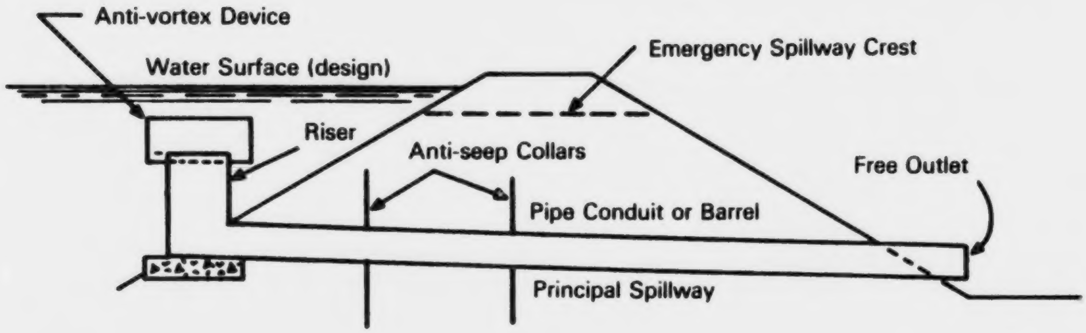
The removal of suspended solids from waterways is based on the concept of gravitational settling of the suspended material.

A typical design for a sedimentation basin embankment is illustrated in Figure 3-25. The major components include a principal and emergency spillway, an anti-vortex device and the basin.

The principal spillway consists of a vertical pipe or riser jointed to a horizontal pipe (barrel) that extends through the dike and outlets beyond the water impoundment. The riser is topped by the anti-vortex device and trash rack which improve the flow of water into the spillway and prevent floating debris from being carried out of the basin. The riser should be watertight and, except for the dewatering opening at the top, should not have any holes, leaks, or perforations. The riser base should be attached to a watertight connection and have sufficient weight to prevent flotation of the riser.

The water discharged from the sediment basin through the principal spillwAy should be conveyed in an erosion-free manner to an existing stable stream.

FIGURE 3-25.
TYPICAL DESIGN OF A SEDIMENT BASIN EMBANKMENT



EMBANKMENT

Thus, at the discharge end of the spillway pipe, an impact basin, rip-rap, excavated plunge pools, and revetment should be constructed as protective measures against scour.

Emergency spillways are also suggested in the design of a sediment basin. They are provided to convey large flows safely past an earth embankment, and they are usually open channels excavated in earth, rock, or reinforcement concrete.

The size of a sedimentation basin is determined from characteristics of flow such as the particle size distribution for suspended solids, the inflow concentration, and the volumetric flow rate. To calculate the area of the sedimentation basin pond required for effective removal of suspended solids, the following data on the flow characteristics are needed:

- The inflow concentration of suspended solids.
- The desired effluent concentration of suspended solids. The desired effluent concentration is usually regulated by local and/or Federal government authorities.
- The particle size distribution for suspended solids.
- The water flow rate (Q) to the pond. For a pond receiving direct run-off, the run-off volume over a certain period of time must be determined. As an example, EPA has chosen the 10-year, 24-hour precipitation event as a design criteria for the overflow rate determination.

The steps in calculating the required area of the sedimentation basin are as follows:

(1) Calculate the removal efficiency of the pond by using the following formula:

$$R\ (\%\ \text{solids removed}) = 1 - \frac{10^6/(C_1\ -1)}{10^6/(C_2\ -1)} \times 100$$

Where:

C_1 = solids concentration in influent (mg/l)
C_2 = solids concentration in effluent (mg/l)

(2) Determine the smallest size of particle that must be removed to achieve the required removal efficiency. The size of the removed particle can be graphically obtained from a particle size distribution for the suspended solid in the influent to the pond. Figure 3-26 shows a typical "particle size distribution" graph.

(3) Once the particle size is chosen, the settling velocity associated with the selected particle size can then be calculated using Stoke's law:

$$V_s = \frac{g}{18\mu}(S-1)D^2$$

where:

V_s = settling velocity (cm/sec)
g = gravitational acceleration (981 cm/sec^2)
= kinematic viscosity of the fluid (cm/sec^2)
s = the specific gravity of the particle
D = diameter of the particle (cm), assumed to be a sphere

(4) With the obtained water flow rate to the pond and the settling velocity, surface area of the pond can then be calculated as follows:

$$A = \frac{Q}{V_s}$$

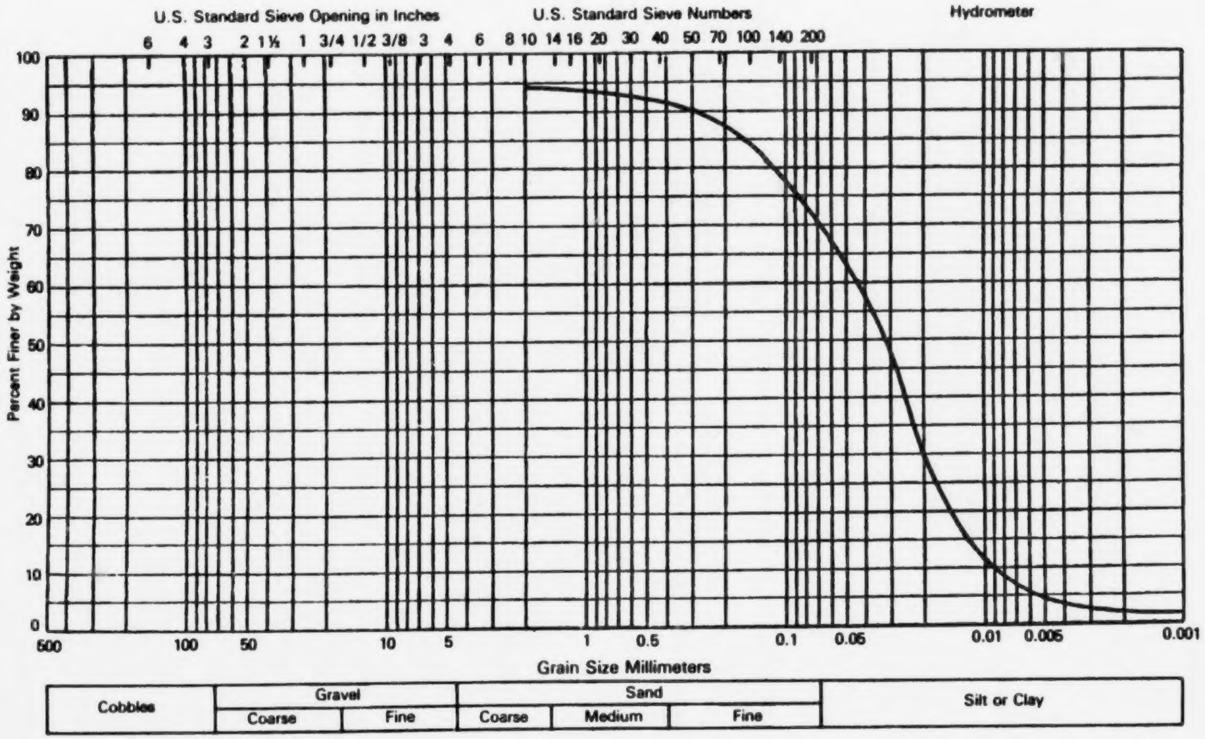
where:

A = required area of the pond (m^2)
Q = the volumetric flow rate thorugh the pond (overflow rate) (m^3/sec)
V_s = the critical settling velocity (m/sec)

(5) The final step is to multiple the required area of the pond by a safety factor of 1.2 to account for non-ideal settling:

$$A_{\text{adjusted}} = 1.2\ A_{\text{required}}$$

FIGURE 3-26.
PARTICLE SIZE DISTRIBUTION GRAPH



The efficiency of sedimentation ponds varies considerably as a function of the overflow rate. Sedimentation ponds perform poorly during periods of heavy rains and cannot be expected to remove the fine-grained suspended solids (Rogoshewski, 1978). If the sedimentation pond is expected to remove sediments that may have been contaminated by waste materials, consideration should be given to improving removal efficiencies by modifying basin or outlet design. One such modification to the outlet structure is shown in Figure 3-27. It is essentially a dual media sand filter surrounding the riser pipe. Other possible design modifications include the use of baffles, extra wide inflow or outflow weirs, energy dissipators, and siphon drawdown riser pipes (Rogoshewski, 1978). Alternatively, a two-pond system could be considered that should significantly increase removal efficiencies.

The quantity of material to be stored is also an important consideration in the construction of the sedimentation basin. The required storage capacity can be calculated by multiplying the total area disturbed by a constant sediment yield rate. Table 3-11 shows the storage requirements for some States. In many States, there is a requirement to clean the sedimentation basin when the sediment accumulation reaches a specified limit imposed by the State. As an example, in West Virginia, the law requires cleaning of the sedimentation pond when the level of accumulation is 60 percent of the design capacity.

3.5.6.4 Construction/Implementation Considerations

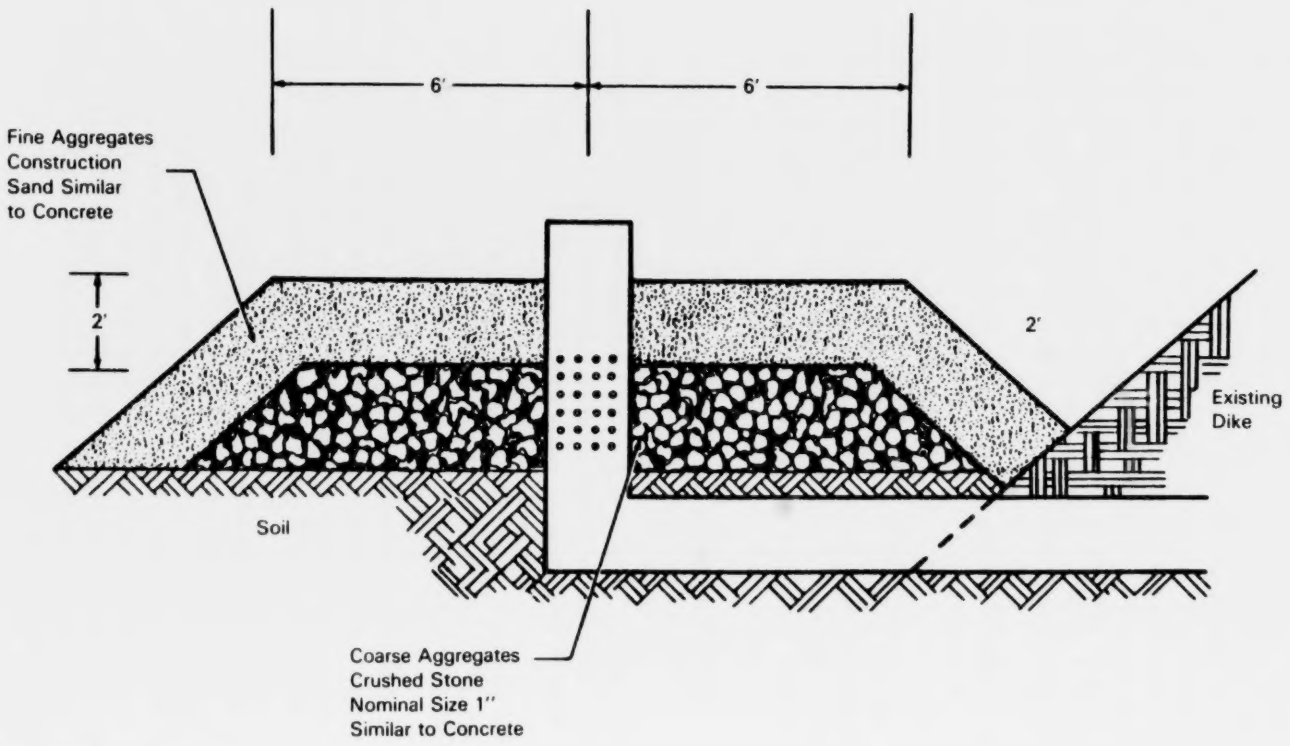
Prior to the construction the areas under the embankment and any structural works shall be cleared, grubbed, and stripped of topsoil to remove trees, vegetation, roots, or other objectionable material.

Fill material for the embankment should be clean mineral soil, free of roots, woody vegetation, oversized stones, rocks, or other objectionable material. Areas on which fill is to be placed should be scarified prior to placement of fill. The fill material should contain sufficient moisture so that it can be formed by hand into a ball without crumbling to facilitate proper compaction. Compaction shall be obtained by routing the hauling equipment over the fill so that the entire surface of the fill is traversed by at least one wheel or tread track of the equipment, or by using a compactor.

The riser of the principal spillway should be securely attached to the barrel by a watertight connection and the barrel and riser should be placed on a firm compacted soil foundation. The base of the riser should be firmly anchored to prevent floating. Previous materials such as sand, gravel, or crushed stone should not be used as backfill around the barrel. Fill material (minumum of 2 feet) is placed around the pipe in thin layers and compacted by hand at least to the same density as the embankment before crossing it with construction equipment.

Design elevations, widths, entrance and exit channel slopes are critical to the successful operation of the spillway and should be adhered to closely during construction.

FIGURE 3-27.
MODIFICATION OF CONVENTIONAL SEDIMENTATION POND
TO REDUCE SUSPENDED SOLIDS



Source: Rogoshewski, 1978

TABLE 3-11.
DESIGN STORAGE CAPACITY REQUIREMENTS FOR SEDIMENTATION BASINS

State	Requirement
Maryland	0.5 inches/acre drained 0.2 inches/acre drained[1]
Kentucky	0.2 acre-ft/acre disturbed
West Virginia	0.125 acre-ft/acre disturbed (60 percent[2])
Pennsylvania	$V = (AIC) + (AIC/3)$ V = volume (ft^3) A = area drained I = rainfall/24 hours C = run-off constant

[1] To be cleaned when storage capacity drops below 0.2 inches/acre drained.

[2] To be cleaned when sediment accumulation approaches 60 percent design capacity.

Source: USEPA, 1976

The embankment and emergency spillway of the sediment basin should be stabilized promptly with temporary vegetation (Virginia SWCC, 1980).

3.5.6.5 Operation, Maintenance, and Monitoring

The embankment of the basin should be checked regularly to ensure that it is structurally sound and has not been damaged by erosion or construction equipment. The emergency spillway should also be checked regularly to ensure that its lining is well established and erosion-resistant. The basin should be checked after each runoff-producing rainfall for sediment cleanout.

3.5.6.6 Costs

The costs for sedimentation basins vary with size, location, and construction method. Table 3-13 at the end of this chapter lists costs for equipment, materials, and construction needed for installation of sedimentation basins.

3.5.7 Levees and Floodwalls

3.5.7.1 General Description

Levees are earthen embankments that function as flood protection structures in areas subject to inundation from tidal flow or riverine flooding. Levees create a barrier to confine floodwaters to a floodway and to protect structures behind the barrier. Floodwalls perform much the same function as levees, but are constructed of concrete.

For hazardous waste sites, levees and floodwalls help to control major losses of waste and cover material and prevent massive leachate producton and subsequent contamination from riverine or tidal flooding.

3.5.7.2 Applications/Limitations

Flood containment levees are most suitable for installation in flood fringe areas or areas subject to storm tide flooding, but not for areas directly within open floodways (Tourbier and Westmacott, 1974).

Flood containment levees may be constructed as perimeter embankments surrounding disposal sites located in floodplain fringe areas, or they may be installed at the base of landfills along slope faces that are subject to periodic inundation. Levees serve to protect land disposal sites from floodwaters, which may erode cover materials and transport waste materials off-site, or which may add water to waste materials and thus increase hazardous leachate production.

Because of the relatively long, flat side slopes of levees, an embankment of any considerable height requires a very large base width. For locations with limited space and fill material, or excessive real estate costs, the use of concrete floodwalls is preferred as an alternative to levee construction.

3.5.7.3 Design Considerations

To provide adequate flood protection, levees should be constructed to a height capable of containing a design flood of 100-year magnitude. Levees designed to protect disposal areas from maximum flood levels of lesser magnitude/greater frequency (e.g., 50-year flood) may not provide sufficient flood protection, particularly for sites known to contain large quantities of hazardous wastes. Elevation of 100-year base flood crests can be determined from floodplain analyses typically performed by state or local flood control agencies.

Figure 3-28 presents design features of a typical levee constructed at the toe of a landfill slope. This design is appropriate for new or uncompleted disposal sites; filled wastes may eventually be placed on the inboard slope of the levee. Where levee construction is impractical due to lack of soil or limited space, perimeter protection of vulnerable landfill slopes may be afforded by the design shown in Figure 3-29. A minimum top width of 10 feet is recommended for most levees; this will permit easy access for construction and maintenance equipment (Linsley and Franzini, 1979).

Floodwalls are designed to withstand the hydrostatic pressure exerted by water at the design flood level. They are subject to flood loading on one side only; consequently, they need to be well founded (Tourbier and Westmacott, 1974). Figure 3-30 presents typical floodwall sections. Like levees, floodwalls may require subsurface cut-offs and interior drainage structures to handle excessive seepage or backwater flow.

3.5.7.4 Construction/Implementation Considerations

Levees are generally constructed of compacted impervious fill. Special drainage structures are often required to drain the area behind the embankment. Ideal construction of levees is with erosion-resistant, low permeability soils, preferably clay. Most levees are homogeneous embankments; but if impermeable fill is lacking, or if seepage through and below the levee is a problem, then construction of a compacted impervious core or sheet-pile cut-off extending below the levee to bedrock (or other impervious stratum) may be necessary. Figure 3-31 depicts these two special cases. Excess seepage through the levees should be collected with gravel-filled trenches or tile drains along the interior of the levee. After draining to sumps, the seepage can be pumped out over the levee. Levee bank slopes, especially those constructed of less desirable soils (silt, sands), should be protected against erosion by sodding, planting of shrubs and trees, or use of stone rip-rap (Linsley and Franzini, 1979).

Fill material used in levee construction should be compacted in layers, with the least pervious layer along the river side of the levee. Because the use of levees will reduce floodplain storage capacity, fill material should be dredged from borrow pits within the floodplain to provide alternate storage volume for floodwaters. This measure will help control rising floodpeaks and

FIGURE 3-28.
TYPICAL LEVEE AT BASE OF DISPOSAL SITE

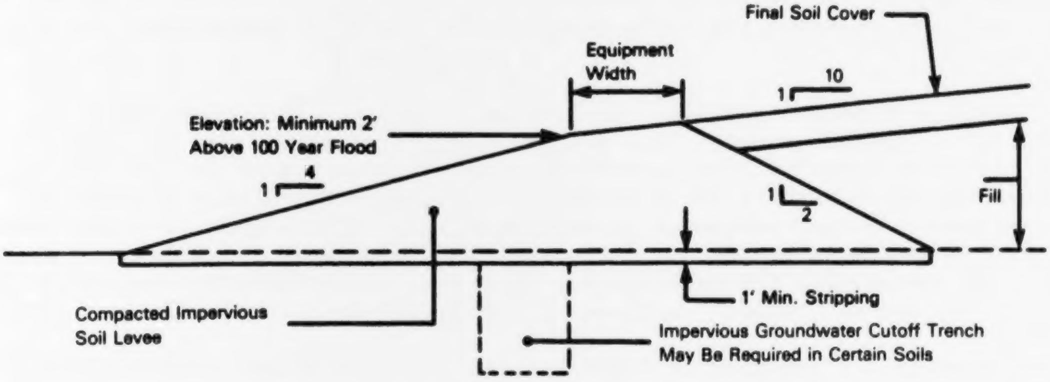

Final Soil Cover
Equipment Width
1
10
Elevation: Minimum 2' Above 100 Year Flood
1
4
1
2
Fill
1' Min. Stripping
Compacted Impervious Soil Levee
Impervious Groundwater Cutoff Trench May Be Required in Certain Soils

FIGURE 3-29.
PERIMETER FLOOD PROTECTION STRUCTURE

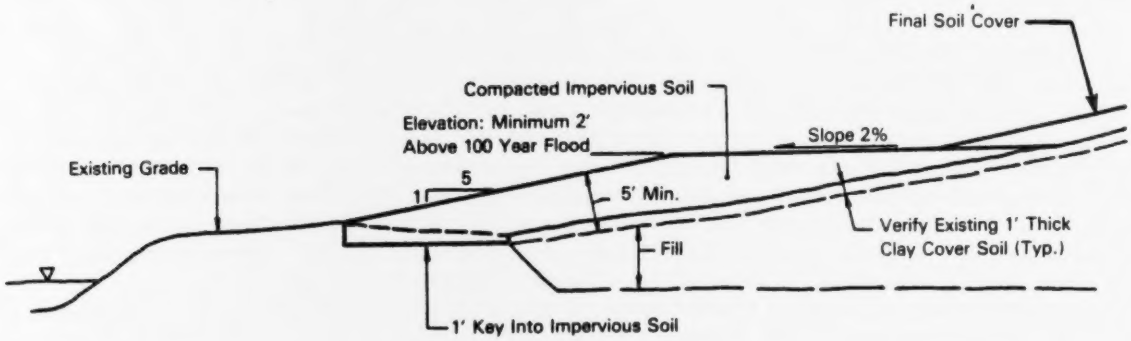
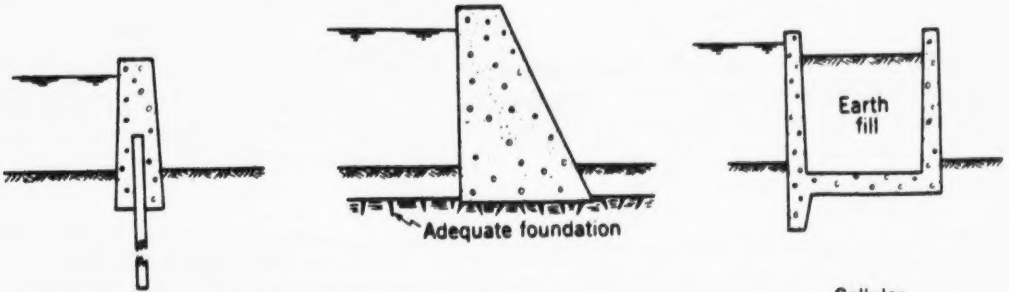

Final Soil Cover
Compacted Impervious Soil
Elevation: Minimum 2' Above 100 Year Flood
Slope 2%
Existing Grade
1
5
5' Min.
Verify Existing 1' Thick Clay Cover Soil (Typ.)
Fill
1' Key Into Impervious Soil

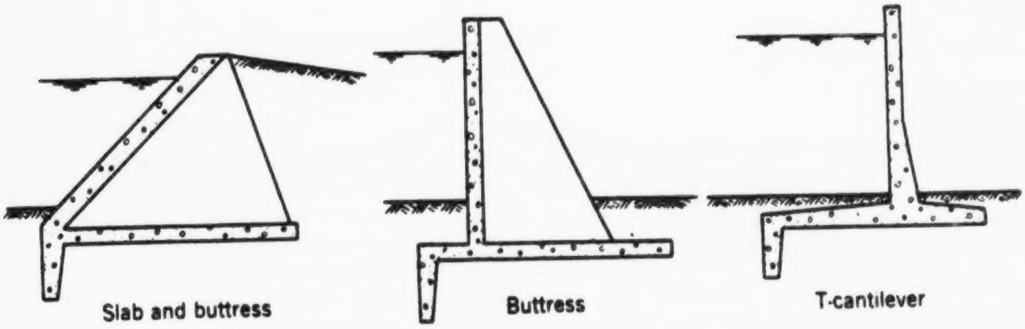
FIGURE 3-30.
SOME TYPICAL FLOODWALL SECTIONS



Simple cantilever and sheet pile

Gravity

Cellular



Slab and buttress

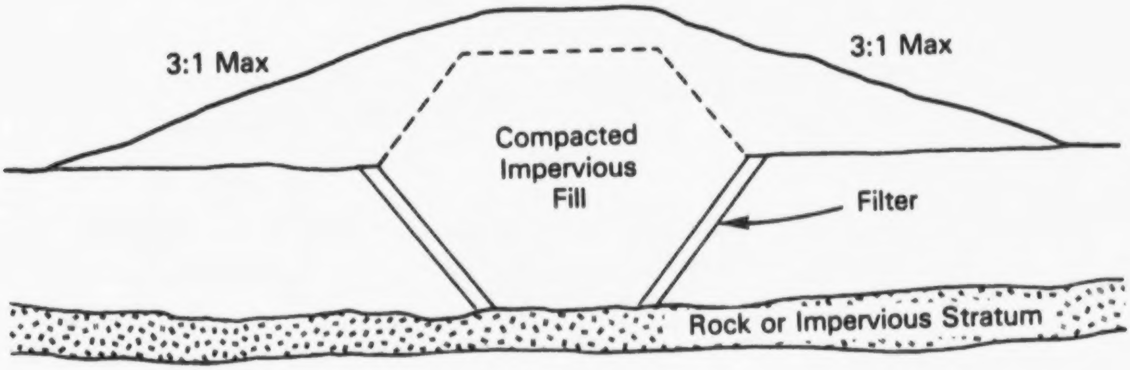
Buttress

T-cantilever

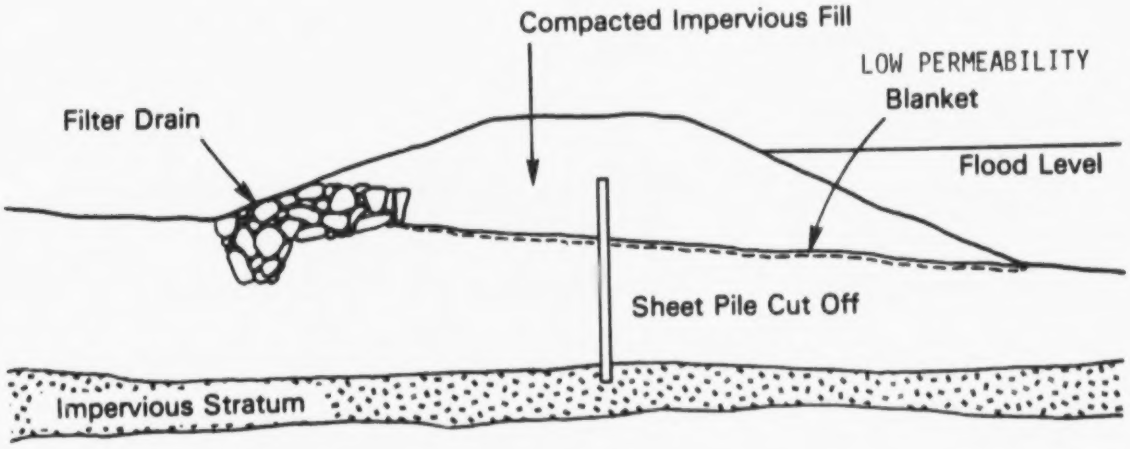
Source: Linsley and Franzini, 1979

FIGURE 3-31.
LEVEE CONFIGURATIONS

Levee with Impervious Core



Levee with Cut-Off and Drain



Source: Tourbier and Westmacott, 1974

prevent an increase in depth of downstream flood stage (Tourbier and Westmacott, 1974).

Storm run-off from precipitation falling on the drainage area behind the levee may cause backwater flooding. To handle such interior drainage, upslope interceptor ditches, diversions, or grassed waterways can be used to channel run-off to downslope holding basins (for subsequent pumping) or to off-site streams for natural gravity discharge. Another method to handle backwater flow is the installation of pressure conduits (with upslope intake works) that discharge beneath the levee. These conduits should be equipped with tidal gates or backwater valves to prevent back-flow and regulate discharge.

3.5.7.5 Operation, Maintenance, and Monitoring

Levees are normally constructed for long-term flood protection but they require periodic inspection and maintenance to assure proper functioning. It is especially important that levees be inspected during and after storm activity.

3.5.7.6 Costs

Costs associated with constructing and maintaining levees will depend on site-specific design variables, availability of suitable embankment soil, and the local frequency and magnitude of flooding. If backwater flooding or seepage is a problem, then special structures must be included in the construction plan. Regular annual inspection for evidence of bank caving, bank sloughing, erosion, and foundation settlement will also increase associated costs. Unit costs relating to levee construction and maintenance are included in Table 3-13 at the end of this chapter.

3.5.8 Selection/Evaluation Criteria

The surface water diversion and containment methods described in this section include a variety of well-established methods. Many of these methods are intended for short-term use as indicated in Table 3-12, and are neither effective nor reliable for use as long-term remedial measures. All of the diversion and containment methods described require frequent inspection, maintenance, and performance checks to ensure continuous reliability. Weekly or even daily inspection may be required for temporary measures, particularly during periods of heavy rainfall. Permanent measures require somewhat less frequent inspections. Operation and maintenance requirements for these measures are relatively simple (e.g., mowing grasses, removing sediments). However, failure of such surface control measures as floodwalls can be costly.

Proper design is critical to the effectiveness and reliability of diversion and collection methods. This often requires the use of several

TABLE 3-12.
NORMAL DURATION OF SURFACE WATER DIVERSION AND COLLECTION MEASURES

Technology	Duration of Normal Use
Dikes and Berms	Temporary
Channels (earthen and CMP)	Temporary
Waterways	Permanent
Terraces and Benches	Temporary and Permanent
Chutes	Permanent
Downpipes	Temporary
Seepage Ditches and Basins	Temporary
Sedimentation Basins	Temporary
Levees	Temporary
Floodwalls	Permanent

control measures in an integrated system. Though detailed design work is often not necessary for these systems, qualified engineering and proper construction practices are imperative in implementing effective controls.

All of the methods described in this section can be easily and rapidly implemented in the field using readily available equipment and materials and local contractors. It is important to ensure that only proper materials and qualified, experienced contractors are used. Improperly installed systems can be detrimental to the integrity of a waste site. Construction seldom poses significant risk to worker health and safety.

The beneficial results from surface water controls are almost immediately evident and for this reason they are frequently implemented as emergency measures to contain and control run-off.

3.5.9 Costs

The costs of excavation, hauling, backfill, grading (spreading, compaction), vegetative stabilization, and maintenance are common to almost all surface flow control techniques. Other costs associated with surface water diversion/collection structures include those for ditch linings, rip-rap slope protection, soil testing, corrugated metal pipe, sheet piling, backflow valves, and sumps. Table 3-13 summarizes these unit costs and the structure construction they apply to.

All cost estimates should be determined on a site-specific basis, considering the specific structures to be installed, all associated earthwork, and any special appurtenances that may be required. A general methodology for estimating costs for construction of surface water diversion/collection structures will contain the following elements:

- Determine source of required earth fill (on-site vs. off-site) and hauling distances
- Determine amount of fill required (cubic yards)
- Determine type and quantity of other materials required (cubic yards of pipe, square feet of rip-rap, etc.)
- Determine costs of installation or placement of these materials using unit costs
- Determine costs of required stabilization for earthen structures (levees, berms, etc.) based on area in square yards to be stabilized; revegetation, rip-rap, or gravel stabilization
- Determine required maintenance or repair costs for a given time period based on reasonable assumptions; for example: assuming the diversion requires rebuilding (new fill and compaction) two times/year after major storms, then costs will be....
- Add all calculated costs for total estimated construction and maintenance expenditures.

Costs are derived simply by multiplying unit costs (shown in Table 3-13) by required quantities of the material or service. These costs will give gross estimates only; they are to be used as general guides for the decisionmaker in evaluating alternative remedial action strategies.

TABLE 3-13.
1985 UNIT COSTS ASSOCIATED WITH SURFACE WATER DIVERSION AND COLLECTION STRUCTURES

Description	Applicable Structures[1]	Cost	Source of Cost Data
Excavation, hauling, grading (spreading and compaction)	All	See Table 3-3	
Trench excavation	D/B; D/D/W; BT; L; DT/B		
• Loam, sand, and loose gravel			
1-6 ft deep, 1/2:1 sides		$0.84 - $1.00/yd^3	2
6-10 ft deep		$0.84 - $0.99/yd^3	2
• Compacted gravel and till			
1-6 ft deep, 1/2:1 sides		$0.84 - $1.18/yd^3	2
6-10 ft deep		$0.84 - $1.06/yd^3	2
Building embankments; spreading, shaping, compacting	All		
• Material delivered by scraper		$0.42 - $0.83/yd^3	2
• Material delivered by back dump		$0.89 - $1.31/yd^3	2
Placement of ditch liner pipe, 1/3 section	D/D/W; drainage benches; C/D		
15 in. radius		$15/ft	2
18 in. radius		$20/ft	2
24 in. radius		$30/ft	2

(continued)

TABLE 3-13. (continued)

Description	Applicable Structures[1]	Cost	Source of Cost Data
Catch basin sump, 3 ft x 4 ft x 1.5 ft	L; DT/B	$217.77 each	2
Corroguated galvanized steel underdrain pipe, asphalt-coated, perforated;	DT/B		
12 in. diameter, 16 gauge		$21/ft	2
18 in. diameter, 16 gauge		$30/ft	2
Corrugated galvanized metal pipe, with paved invert	C/C; SB		
18 in. diameter, 14 gauge		$34.12/ft	2
36 in. diameter, 12 gauge		$86.83/ft	2
48 in. diameter, 12 gauge		$65.12/ft	2
Steel sheet piling;	L (seepage control)		
15 ft deep, 22 lb/ft^2		$\$10.65/\text{ft}^2$	3
20 ft deep, 27 lb/ft^2		$\$12.35/\text{ft}^2$	3
25 ft deep, 38 lb/ft^2		$\$15.70/\text{ft}^2$	3
Backflow preventer; gate valves, automatic operation, flanged, 10" diameter	L (drainage control)	$9,250 each	3
Floating baffles	SB	$15 - $50/ft	4
Sump pumps, 10 ft head, automatic	L (backwater drainage)		
Bronze		$25 - $505 each	3
Cast Iron		$145 - $295 each	3

(continued)

TABLE 3-13. (continued)

Description	Applicable Structures[1]	Cost	Source of Cost Data
Revegetation, mulching, maintenance	D/B; D/D/W; BT; L	See Table 3-4	
Loose gravel, excavation, loading, hauling 5 miles, spreading and compacting	All (slope protection; drainage)	\$8 - \$8.50/yd^3	2
Stone rip-rap; dumped from trucks, machine-placed	All (slope protection; channel & outlet stabilization)	\$21/yd^3	3
Soil testing;	All (preconstruction evaluation)		
liquid and plastic limits		$30/test	3
hydrometer analysis specific gravity		$50/test	3
moisture content		$7.15/test	3
permeability		$65/test	3
Proctor compaction		$110 - $120/test	3
shear tests			
triaxial		$150 - $280/test	3
direct shear		$95 - $260/test	3
Temporary Diversion Dike	D/B	$1.31 - $2.62/ linear foot	5
Temporary sediment construction, drainage area:			
1-25 acres		$394 - $2,082	5
50-75 acres		$3,942 - $6,470	5
75-100 acres		$6,470 - $8,340	5
100-125 acres		$8,340 - $10,913	5

(continued)

TABLE 3-13. (continued)

Description	Applicable Structures[1]	Cost	Source of Cost Data
Sediment removal from basins	SB	\$3.95 - \$9.10/yd[3]	5
Paved flume, installed	C/D; SB	\$26.30 - \$39.92/yd[2]	5
Level spreader construction	D/B; D/D/W; BT; C/D	\$3.23 - \$6.47/ linear foot	5

[1] Key: D/B, dikes and berms; D/D/W, ditches, diversions, and waterways; BT, bench terraces; C/D, chutes and downpipes; L, levees; DT/B, drainage trenches and basins; SB, sediment basins.

[2] McMahon, 1984

[3] Godfrey, 1984

[4] Klikas, C., Environetics, Inc., Bridgeview, IL, personal communication, 1985.

[5] Virginia SWCC, 1980. Updated to 1985 dollars using Engineering News-Record Construction Index.

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SECTION 4

AIR POLLUTION CONTROLS

Air pollution problems at uncontrolled waste sites are the result of gaseous emissions and fugitive dusts.

Gases may be emitted by the vaporization of liquids, venting of entrained gases or by chemical and biological reactions with solid and liquid waste material. Volatile organics may be released slowly but continuously from surface impoundments or landfills.

Fugitive emissions are particulates that are lifted from the ground by means of one or more of the following processes:

- Wind erosion of exposed waste materials or cover soil
- Re-entrainment of particulate matter by vehicular traffic on haul roads and exposed surfaces
- Excavation of waste materials during remedial action.

4.1 Control of Gaseous Emissions to the Atmosphere

Methods for controlling the release of gaseous emissions to the atmosphere include covers for control of volatile emissions from impoundments and active gas collection systems for collection and control of gases generated in landfills.

4.1.1 Covers

4.1.1.1 General Description

Covers involve the placement of a barrier at the water-air interface to reduce gaseous emissions. Lagoon covers, floating immiscible liquids and floating spheres can all be used for this purpose.

Floating lagoon covers consist of a synthetic lining placed in one piece over an impoundment with proper anchoring at the edges and floats to prevent the lining from submerging. Floating lagoon covers have been described fully in Section 3.2.

Floating immiscible liquids include a variety of water insoluble substances, particularly long chain aliphatic alcohols, which form a monolayer on the surface of an impoundment, and thereby inhibit the volatilization of water soluble compounds (Vogel, 1985). Compounds that can be used for this purpose include: buryl alcohol, hexadecanol and polydimethylsiloxane (Vogel, 1985; Valsaraj et al., 1985).

A third type of cover material involves the use of floating polypropylene spheres which arrange themselves into a closely packed configuration and thereby reduce emissions of volatile compounds by reducing the exposed surface area.

4.1.1.2 Appliations/Limitations

The various cover materials described above provide temporary methods for reducing volatile emissions from surface impoundments.

Floating lagoon covers function as both a surface water control mechanism as well as a mechanism for controlling gaseous emissions; they prevent overtopping of the impoundment while retarding volatilization of hazardous compounds. Performance of floating lagoon covers is only minimally affected by wind. They are suitable to situations where more than a year will elapse before final closure of the lagoon. They are not suitable for lagoons with weak berms or for lagoons located in areas that cannot support heavy construction equipment. A variety of cover materials are available and the materials should be selected based on chemical resistance (see Section 3.2).

Floating polyethylene spheres are capable of reducing volatile emissions by up to 90 percent. Polyethylene spheres are compatible with a broad range of compounds including inorganic acids and bases and most aromatic and aliphatic organic compounds (Vogel, 1985).

Floating immiscible liquids are suitable for controlling emissions of water soluble organics only (Vogel, 1985). Their effectiveness is dependent on the formation of a continuous monolayer on the impoundment surface. The control efficiency for a surface impoundment containing a floating layer of organics is poor since this prevents formation of the monolayer. Wave action also destroys monolayer effectivenes (Vogel, 1985). Effectiveness is temporary, and the maximum desirable interval between applications is estimated to be one to two weeks (Vogel, 1985).

4.1.1.3 Technology Selection/Evaluation

Floating covers should be considered whenever there is a need for temporary control of volatile emissions from a surface impoundment. The available technologies range from use of low cost, floating immiscible liquids which can be easily emplaced on a surface impoundment for immediate, short-term emissions control to floating lagoon covers which are considerably more costly and difficult to install, but result in more reliable and long-term emissions control.

4.1.1.4 Cost

Costs for floating lagoon covers are addressed in Section 3.2.

Floating spheres are priced according to their diameter. Table 4- shows approximate 1985 costs for various diameter spheres and the approximate number of spheres needed per square foot of coverage.

Table 4-1. Cost and Coverage for Several Sizes of Polypropylene Spheres

Sphere diameter (mm)	Bulk Price	Number required per ft^2 coverage	Cost of coverage \$ per ft^2
20	\$24/1000	230-280	5.52-6.72
38	\$117/800	75-85	10.97-12.41
45	\$46.50/500	45-55	4.19-5.12
145	\$75/50	4-5	6.00-7.50

Source: Capricorn Chemical Corp., Secaucus, N.J., personal communication, July 1985.

The cost of surfactant layers depends upon the particular surfactant used and the required thickness of the monolayer. For example, only 2×10^{-5} lb of hexadecanol would be required to form a suitable monolayer for control of highly water soluble compounds. The chemical cost of the monolayer would be approximately 0.03 cents per square foot per application.

4.1.2 Active Interior Gas Collection/Recovery System

4.1.2.1 General Description

An active interior gas collection system alters the pressure gradients and paths of gas migration by mechanical means. This type of system is described in detail in Section 6.3. It typically consists of four components: gas extraction wells, gas collection headers, vacuum blowers or compressors and a treatment system. The centrifugal blowers create a vacuum through the collection headers and wells to the wastes and the ground surrounding the wells. A pressure gradient is thereby established, inducing flow from the landfill (which is normally under positive pressure) to the blower, creating a negative, or vacuum pressure. Subsurface gases flow in the direction of decreasing pressure gradient (through the wells, the header, and the blower) and are disposed of by treatment (SCS, 1980).

4.1.2.2 Applications/Limitations

Active interior gas collection/recovery systems are used to collect gases from beneath a landfill surface before they are vented to the atmosphere. They can be installed at virtually any site where it is possible to drill or excavate through the landfilled material to the required depth. Limiting factors could include the presence of free-standing leachate (i.e., saturation) or impenetrable materials within the landfill.

4.2 Fugitive Dusts

Commonly used measures for controlling fugitive dusts from inactive waste piles and active cleanup sites include use of chemical dust suppressants, wind screens, water spraying and other dust control measures commonly used during construction. Each is described below.

4.2.1 Dust Suppressant

4.2.1.1 General Description

Dust suppressants include a wide range of natural and synthetic waste materials which strengthen bonds between soil particles and hold this strengthened condition for an appreciable period of time. A wide variety of resins, bitumenous materials and polymers are marketed as dust suppressants. Table 4-2 lists a variety of commerically available formulations along with

Table 4-2. Commercially Available Dust Suppressant Costs, and Recommended Application Rates

Name	Material cost/acre, (1985 $)	Formulation	Application concentration*	Application rate
Soil Seal	1450	Latex acrylic copolymer	3%	1.0 gal/yd^2
AMSCO-RES 4281	2350	Carboxylated styrene-butadiene copolymer	20%	0.6 gal/yd^2
Supac 8 NP	4840	Nonwoven geotextile	8 oz/yd	12-ft folls
Flambinder	62	Lignosulfonate	17%	0.5 gal/yd^2
Genaqua	484	Vinyl acetate resin	10%	0.2 gal/yd^2
Curasol	273	Synthetic resin	3%	0.3 gal/yd^2
M166 & M167	891	Latex	7%(MI66) + 0.2%(MI67)	0.5 gal/yd^2
Coherex CRF	847	Petroleum resin	25%	0.5 gal/yd^2
Sherman Process (no grass seed)	800	Straw mulch with emulsified asphalt	NA	NA
Sherman Process (with grass seed)	800	Straw mulch with emulsified asphalt	NA	NA
Terra Track I	68	Vegetable gum	0.3%	1.4 gal/yd^2

NA = not applicable.
*Percent compound in aqueous solution, unless otherwise indicated.
Source: Rosbury and James, 1985.

their approximate costs and application rates which were used to test product effectiveness.

Chemical dust supressants are most commonly applied with water wagons equipped with two to five nozzles that shoot a flat spray behind the vehicle. The flow-control system is often crude and difficult to regulate, and it is not usually tied to vehicle speed. A calibrated spray bar is more suitable for application of chemical dust suppressants. Sophisticated systems exist which allow the operator to specify an application rate and the truck will automatically regulate the speed and spray rate. Some bitumens must be applied with an asphalt distributor because the material must be heated before application (USEPA, 1985).

4.2.1.2 Applications/Limitations

Dust suppressants are used primarily to temporarily bind soil particles and reduce fugitive dust emissions from inactive waste piles. Effectiveness of a dust suppressant is dependent upon maintaining the soil-chemical crust. Emerging weeds and any type of disturbance from traffic will break this crust. Preemergent weed control may be used before applying dust suppressant. If undisturbed, dust suppressants can be expected to be 100 percent effective for a period of 1 to 4 weeks using the concentrations shown in Table 4-2, depending upon the formulation. Control efficiencies decline thereafter (Rosbury and James, 1985). Dust suppressants may also be used to control dust from active work areas (e.g., haul roads), although they are less effective for this application and require frequent reapplication.

4.2.1.3 Technology Selection/Evaluation

Use of dust suppressant provides an effective means for temporary control of fugitive dust emissions. They are reliable as a short-term control measure but must be periodically reapplied to provide long-term fugitive dust control. Application is straightforward and can be accomplished quickly. The dust suppressants are simply sprayed on the area of concern. There is the potential for secondary impacts (soil and groundwater contamination) from the use of certain chemical dust suppressants which contain toxic substances.

4.2.1.4 Costs

Table 4-2 summarizes the costs of various dust suppressants. Costs for chemical dust suppressants range from $68/acre for a natural vegetable gum to $2,350/acre for carboxylated styrene-butadiene copolymer. Use of nonwoven geotextile is considerably more costly than chemical dust suppressants. Application of this material for dust control can be expected to cost $4,840/acre.

4.2.2 Wind Fences/Screens

4.2.2.1 General Description

A wind fence is a porous screen which takes up or deflects a sufficient amount of wind so that the wind velocity is lowered below the threshold required for initiation of soil movement. Wind screens are typically 4 to 10 feet high and are composed of polyester or other high strength material.

4.2.2.2 Applications/Limitations

Wind screens are used to reduce wind velocity and control fugitive dusts. Tests conducted by EPA on the efficiency of wind screens in controlling dust from fly ash stockpiles indicated that they were about 60 percent efficient in controlling inhalable particulates (less than 15 microns) at wind speeds of about 10 to 13 mph (with gusts of 18 to 19 mph). Control efficiencies for total suspended particulates were about 75 percent. (Larson, undated). In another study conducted by PEDCo and reported in USEPA (1985), results showed no consistent benefits from windscreen for particles in the less than 10-micrometer respirable size range although larger size particles could be controlled. Maximum reduction of wind velocity can be expected for a distance of 1 to 5 fence heights downstream.

4.2.2.3 Technology Selection/Evaluation

Wind fences offer a low cost method of reducing fugitive dust emissions. The fences can be easily transported and set-up for application in various work areas. However they are, at best, only partially effective in controlling inhalable particulates.

4.2.2.4 Costs

The installed cost of a polyester wind screen is approximately $13 per linear foot for a screen six feet high and ranges from $2.00 to 2.50 per square foot of screen for other heights (Vogel, 1985).

4.3 Water Spraying and Other Commonly Used Techniques

There are a number of other control measures for fugitive emissions which are simple and easy to implement, and do not warrant a detailed discussion.

The most commonly used method for control of dust emissions is to spray water on the exposed surface areas. This method is mainly used to reduce fugitive dusts along active travel paths, excavation areas and from truck boxes loaded with soils. Active travel areas dry quickly and water must be reapplied frequently (about every 2 hours) to maintain effectiveness (Rosbury and James, 1985). Water is applied to the unpaved road surface with a water wagon, or spray bar. The quantity will vary with the road surface material, sunlight, humidity and traffic level (USEPA, 1985).

Spraying of soils in a truck box can be accomplished by installing a frame which supports a line of spray nozzels on either side of the length of the truck. Contaminated soils can then be sprayed as they are loaded on the truck and as they are being dumped. Water spraying is more effective for large grain-size particles although some control of small particle sizes can be expected. Addition of surfactant to the water can be used to increase control efficiencies and reduce water usage (Rosbury and James, 1985).

The effectiveness of chemical dust supressants and water spraying along unpaved roads can be improved by compaction and addition of roadway aggregate. A road surface will not compact unless the range of particle sizes in the aggregate is in the correct proportion. Table 4-3 shows the proper size gradation for an unpaved road.

Table 4-3. Proper Size Gradation for Unpaved Road Surface

Sieve Size	% Passing	Soil Type
1 in.	100	Gravel
3/4 in.	85-100	
3/8 in.	65-100	
No. 4	55-85	
No. 10	40-70	Sand
No. 40	25-45	
No. 200	10-25	Clay, silt

Source: USEPA, 1985

With too much gravel, relatively little dust will occur (until tires grind the gravel down to silt size particles) but the aggregate will be pushed to the side of the road. Any applied dust suppressant will simply pass through the top surface and provide little control. With too much sand, the bearing capacity will be poor, and any dust suppressant that attempts to form a crust will not work because of rutting (USEPA, 1985).

Reentrained dust from paved roads is controlled by removing dirt from the road surface by sweeping, vacuuming or flushing. Unfortunately, all these methods remove coarse particles more successfully than fine particles (USEPA, 1985).

Fugitive dust emissions from excavation activities can be reduced by maintaining a favorable slope and orientation on the waste pile. The slope of the pile in the prevailing wind direction should be less than 10 degrees to minimize emissions. Measured wind velocities will accelerate significantly up slopes greater than 10 degrees, and the zone of maximum particle entrainment is at the top of the pile. If the length of a pile is perpendicular to the prevailing wind direction, emissions can be reduced about 60 percent from a pile with the length parallel to the wind direction (Vogel, 1985).

Dust emissions from waste piles can be substantially reduced by covering the pile. A synthetic cover can be placed over the piles and secured with tension cables. An auger feed system can also be installed to emplace and remove wastes from the pile. The installed cost for such a structure ranges from about $4 to $6 per square foot of pile surface area (Vogel, 1985).

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SECTION 5

GROUNDWATER CONTROLS

Control of groundwater contamination involves one of four options: (1) containment of a plume; (2) removal of a plume after measures have been taken to halt the source of contamination; (3) diversion of groundwater to prevent clean groundwater from flowing through a source of contamination or to prevent contaminated groundwater from contacting a drinking water supply; or (4) prevention of leachate formation by lowering the water table beneath a source of contamination. Remedial technologies for controlling groundwater contamination problems are generally placed in one of four categories: (1) groundwater pumping, involving extraction of water from or injection of water into wells to capture a plume or alter the direction of groundwater movement; (2) subsurface drains, consisting of gravity collection systems designed to intercept groundwater; (3) low permeability barriers, consisting of a vertical wall of low permeability materials constructed underground to divert groundwater flow or minimize leachate generation and plume movement; or (4) in-situ treatment methods to biologically or chemically remove or attenuate contaminants in the subsurface. These technologies can be used singularly or in combination to control groundwater contamination.

Pumping, subsurface drains, and barriers are addressed in this section; in-situ methods are discussed in Section 9. Detailed information on many of these technologies can be found in Leachate Plume Management (USEPA, 1985). Detailed information on barrier walls can also be found in Slurry Trench Construction for Pollution Migration Control (Spooner et al., 1984a) and Compatibility of Grouts with Hazardous Wastes (Spooner, et al., 1984b).

5.1 Groundwater Pumping

5.1.1 Description

Groundwater pumping techniques involve the active manipulation and management of groundwater in order to contain or remove a plume or to adjust groundwater levels in order to prevent formation of a plume. Types of wells used in management of contaminated groundwater include wellpoints, suction wells, ejector wells, and deep wells. The selection of the appropriate well type depends upon the depth of contamination and the hydrologic and geologic characteristics of the aquifer.

5.1.2 Applications/Limitations

Well systems are very versatile and can be used to contain, remove, divert, or prevent development of plumes under a variety of site conditions.

Pumping is most effective at sites where underlying aquifers have high intergranular hydraulic conductivity. It has been used with some effectiveness at sites with moderate hydraulic conductivities and where pollutant movement is occuring along fractured or jointed bedrock. In fractured bedrock, the fracture patterns must be traced in detail to ensure proper well placement.

Where plume containment or removal is the objective, either extraction wells or a combination of extraction and injection wells can be used. Figure 5-1 depicts the use of a line of extraction wells to halt the advance of the leading edge of a contaminant plume and thereby prevent contamination of a drinking water supply. Use of extraction wells alone is best suited to situations where contaminants are miscible and move readily with water; where the hydraulic gradient is steep and hydraulic conductivity high; and where quick removal is not necessary. Extraction wells are frequently used in combination with slurry walls to prevent groundwater from overtopping the wall and to minimize contact of the leachate with the wall in order to prevent wall degradation. Slurry walls also reduce the amount of contaminated water that requires removal, so that costs and pumping time are reduced.

A combination of extraction and injection wells is frequently used in containment or removal where the hydraulic gradient is relatively flat and hydraulic conductivites are only moderate. The function of the injection well is to direct contaminants to the extraction wells. This method has been used with some success for plumes which are not miscible with water. Figure 5-2 illustrates an extraction/injection well system for removal. One problem with such an arrangement of wells is that dead spots (i.e., areas where water movement is very slow or nonexistent) can occur when these configurations are used. The size of the dead spot is directly related to the amount of overlap between adjacent radii of influence; the greater the overlaps the smaller the dead spots will be. Another problem is that injection wells can suffer from many operational problems, including air locks and the need for frequent maintenance and well rehabilitation.

Extraction or injection wells can also be used to adjust groundwater levels, although this application is not widely used. In this approach, plume development can be controlled at sites where the water table intercepts disposed wastes by lowering the water table with extraction wells. In order for this pumping technique to be effective, infiltration into the waste pile must be eliminated and liquid wastes must be completely removed. If these conditions are not met, the potential exists for development of a plume of contaminants. The major drawback to using well systems for lowering water tables is the continued costs associated with maintenance of the system.

FIGURE 5-1.
CONTAINMENT USING EXTRACTION WELLS

Figure 5-1a. Cross-Sectional View

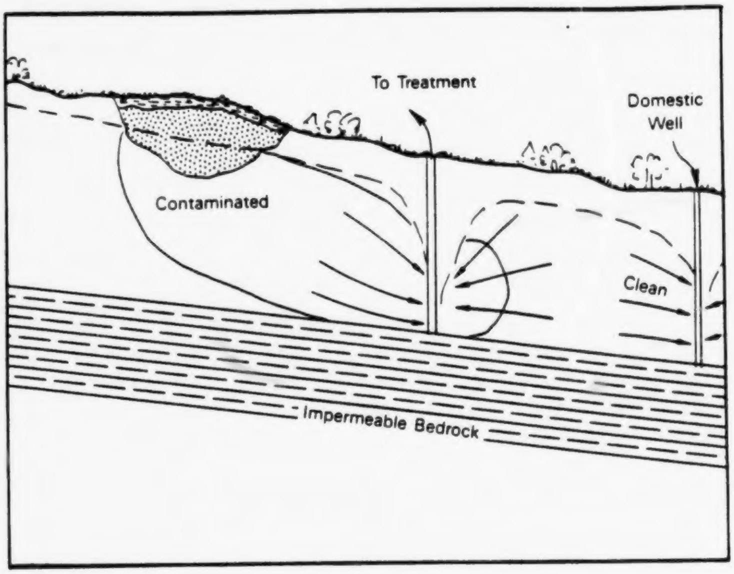
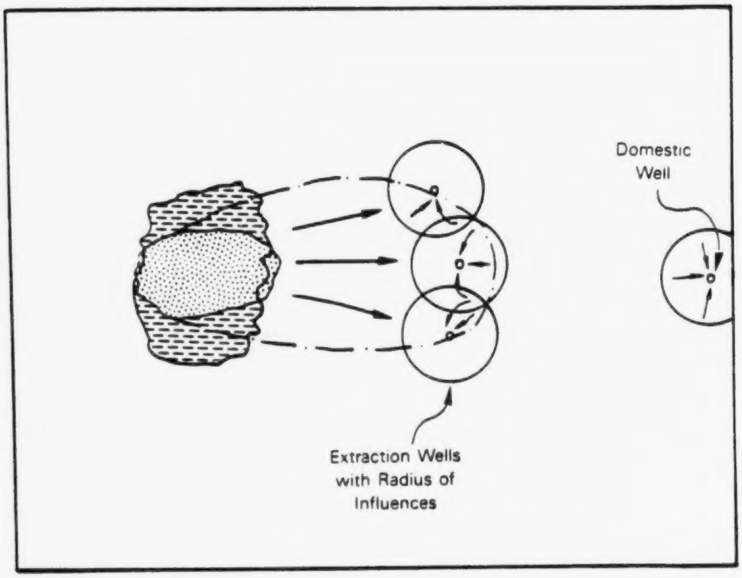
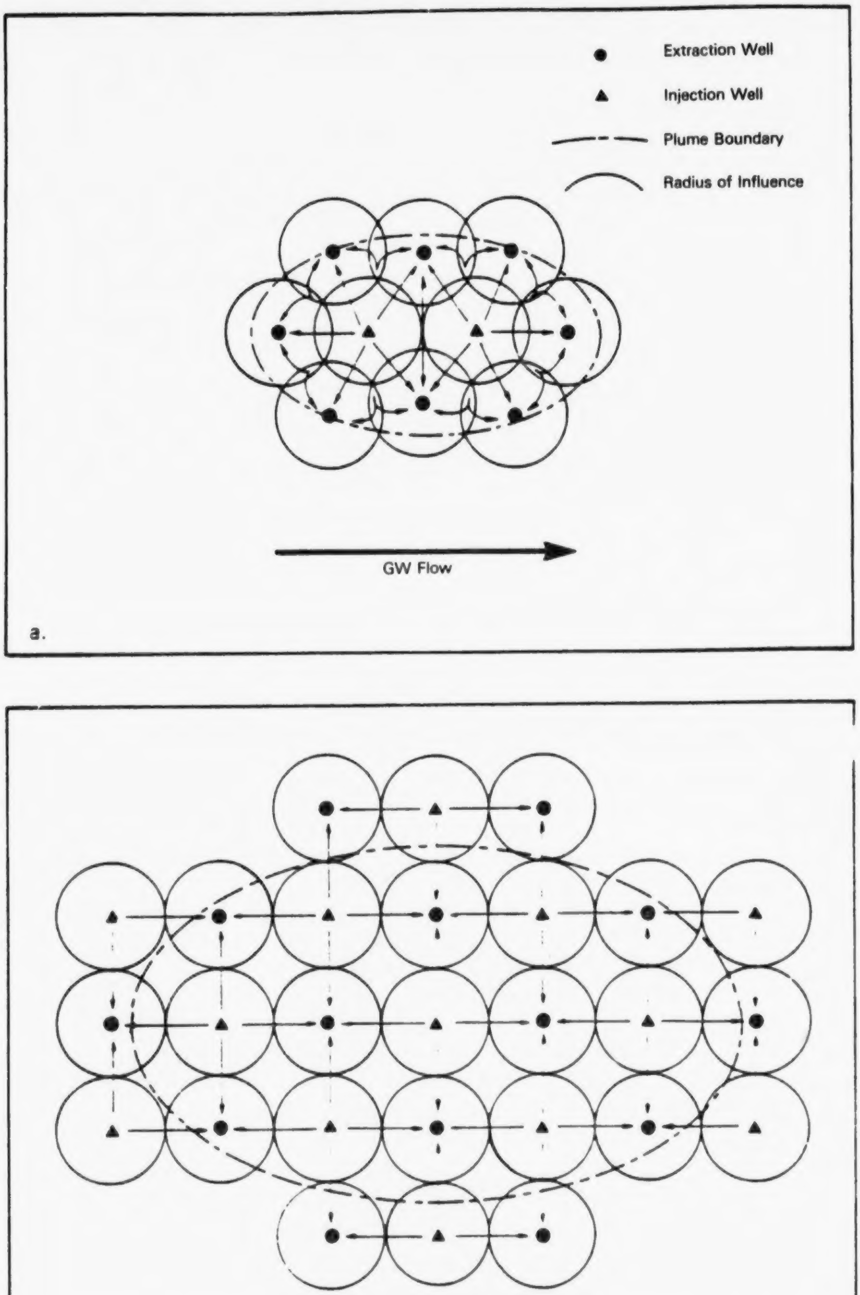


Figure 5-1b. Plan View



Source: USEPA, 1985

FIGURE 5-2.
EXTRACTION AND INJECTION WELL PATTERNS FOR PLUME REMOVAL



Source: USEPA, 1985

Groundwater barriers can be created using injection wells to change both the direction of a plume and the speed of plume migration. Figure 5-3 shows an example of plume diversion using a line of injection wells to protect domestic water sources. By creating an area with a higher hydraulic head, the plume can be forced to change direction. This technique may be desirable when short-term diversions are needed or when diversion will provide the plume with sufficient time to naturally degrade so that containment and removal is not required.

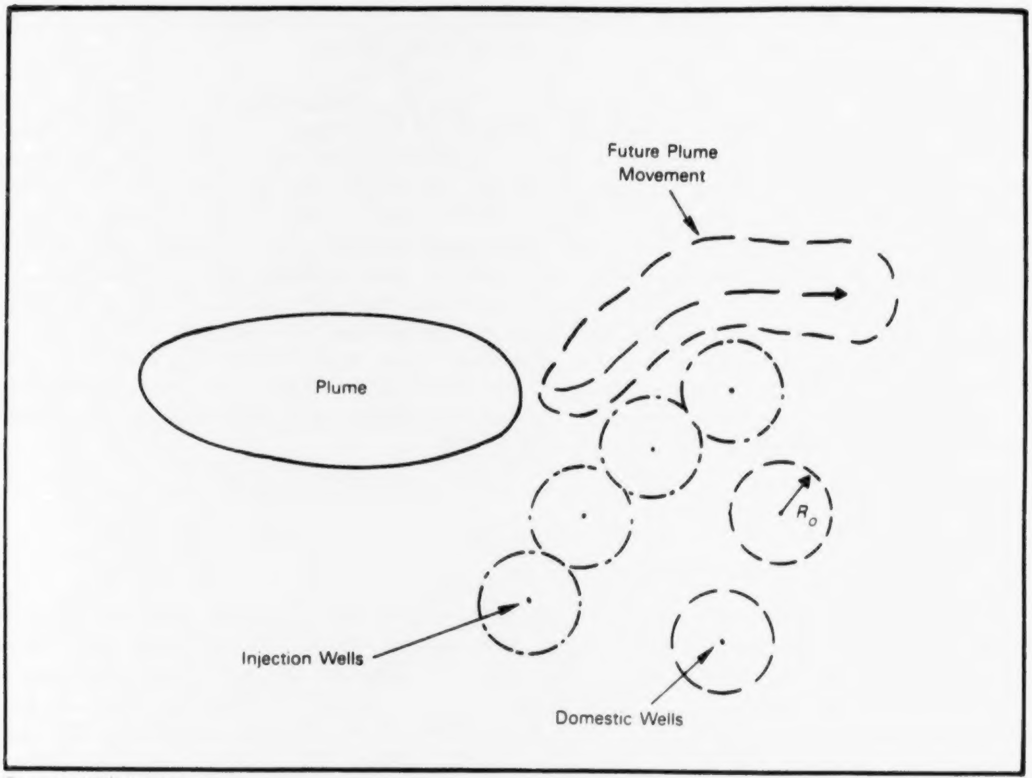
Each of the well types used in groundwater pumping have their own specific applications and limitations as well. Table 5-1 summarizes the site conditions for which each of these well types are most applicable. Section 5.1.3.3 describes the components of these well types.

Wellpoint systems are effective in almost any hydraulic situation. They are best suited for shallow aquifers where extraction is not needed below more than about 22 feet. Beyond this depth, suction lifting (the standard pumping technique for wellpoints) is ineffective. Suction wells operate in a similar fashion to wellpoints and are also depth limited. The only advantage of suction wells over wellpoints is that they have higher capacities. For extraction depths greater than 20 feet, deep wells and ejector wells are used. Deep well systems are better suited to homogeneous aquifers with high hydraulic conductivities and where large volumes of water may be pumped. Ejector wells perform better than deep wells in heterogeneous aquifers with low hydraulic conductivities. A problem with ejector systems is that they are inefficient and are sensitive to constituents in the groundwater which may cause chemical precipitates and well clogging (Powers, 1981).

5.1.3 Design

There are a number of considerations which go into the design of a well system. In this section, design considerations are divided into the following major areas: theory of flow towards a well; design of well systems; and design and selection of well components (e.g., screens, casing, and pumps). Well theory differs for confined and unconfined aquifers. A confined aquifer is a permeable zone between two aquicludes, which are confining beds of clay, silt, or other impermeable materials. Depending on the elevation of the water source, and the permeability and rate of flow in the aquifer, the pressure in a confined aquifer can rise to considerable heights. The pressure will vary considerably depending upon the rate of replenishment, the rate of discharge and other factors. But the quantity of water stored in the aquifer changes only slightly. In an unconfined aquifer, there is no upper confining bed. The water table falls with changing flow conditions in the aquifer. The amount of water stored in the aquifer changes radically with water table movements (Powers, 1981).

FIGURE 5-3.
PLUME DIVERSION USING INJECTION WELLS



Source: USEPA, 1985a

TABLE 5-1.
CRITERIA FOR WELL SELECTION

Parameters	Wellpoints	Suction Wells	Ejector Wells	Deep Wells
Hydrology				
• Low hydraulic conductivities (e.g., silty or clayey sands)	Good	Poor	Good	Fair to Poor
• High hydraulic conductivities (e.g., clean sands and gravel)	Good	Good	Poor	Good
• Heterogeneous materials (e.g., stratified soils)	Good	Poor	Good	Fair to Poor
• Proximate recharge	Good	Poor	Good to Fair	Poor
• Remote recharge	Good	Good	Good	Good
Depth of Well	Shallow <20 ft	Shallow <20 ft	Deep >20 ft	Deep >20 ft
Normal Spacing	5 - 10 ft	20 - 40 ft	10 - 20 ft	>50 ft
Normal Range of Capacity (per unit)	0.1 - 25 gpm	50 -400 gpm	0.1 -40 gpm	25 - 3000 gpm
Efficiency	Good	Good	Poor	Fair

Source: Powers, 1981

5.1.3.1 Theory of Flow Towards a Well

Figure 5-4 illustrates the flow of water towards a well. When pumping is started, the water level in the vicinity of the pump is lowered. The greatest amount of lowering or drawdown is at the well. Drawdown is less at greater distances from the well and at some distance away aquifer lowering is nonexistent. The force or pressure that drives the water towards the well is the head represented by the difference between the water level inside the well and the water level at any place outside the well (H-h/L). The velocity of the groundwater increases as it approaches the well. According to Darcy's law, with increasing velocity, the hydraulic gradient increases as flow converges towards the well. As a result, the lowered water surface develops a continually steeper slope towards the well and this is referred to as the cone of depression. The distance from the center of the well to the limit of the cone of depression is called the radius of influence (R_o). The radius of influence increases with increased hydraulic conductivity and is typically larger for a confined aquifer than it is for an unconfined aquifer.

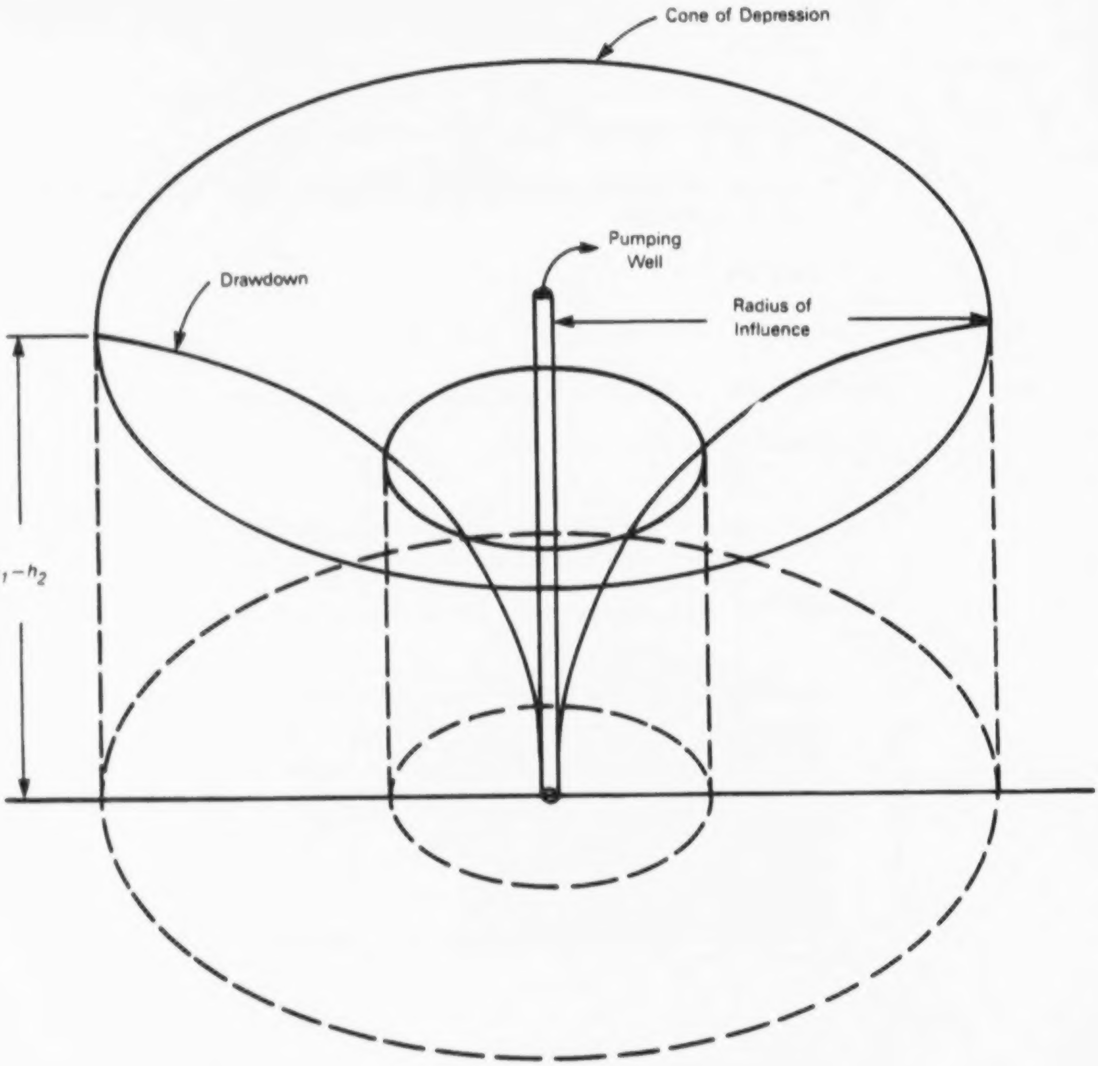
The shape and size of the cone of depression is dependent upon pumping rate, pumping period (i.e., cycles), slope of the original water table, hydraulic barriers, aquifer characteristics, and recharge rates. Two aquifer characteristics that are important in determining the hydraulic characteristics of the water-bearing formation and thereby the cone's configuration are transmissivity (T) and the coefficient of storage (S). Transmissivity indicates how much water will move through a formation and storage (storativity) indicates how much of the water can be removed by pumping. The coefficients of storage and transmissivity are important in being able to make the following predictions: (Johnson Division, UOP, Inc., 1975):

- Specific capacity of wells with differing diameters
- Drawdown of an aquifer at varying distances from the well
- Drawdowns with varying pumping rates and times.

Transmissivity and storativity are typically determined during a pump test.

A number of well formulas have been developed to describe both equilibrium (steady state) and nonequilibrium flow conditions. This section presents equilibrium well formulas in order to illustrate the relationship between various factors (permeability, drawdown, radius of influence, etc.) and provide a simple means for estimating the radius of influence or drawdown when other factors are known. The equations presented below assume ideal aquifer conditions (e.g., uniform permeability, laminar flow, and constant aquifer saturation and thickness). However, such assumptions do not severely limit the use of these formula for approximating flow to a well. They do not account for the effect of hydrologic boundary conditions or partially penetrating wells on flow. These flow conditions will be discussed later in this section.

FIGURE 5-4.
FORMATION OF CONE OF DEPRESSION
FOR A PUMPED WELL



Source: USEPA, 1985

Theim and Forchheimer independently derived equations for equilibrium (steady state) radial flow to wells under confined and unconfined aquifer conditions, as described in Johnson Divison, UOP, Inc. (1975). Both equations assume recharge at the edge of the cone so that the cone dimensions remain constant as long as pumping rates remain constant.

The basic formula for an unconfined aquifer (shown in Figure 5-5) is:

$$H^2 - h_w^2 = (Q/\pi k) \ln (R_o/r_w) \tag{5-1}$$

where:

H = saturated thickness of the aquifer (ft)

h_w = height of water at the well (ft) measured from the bottom of the aquifer

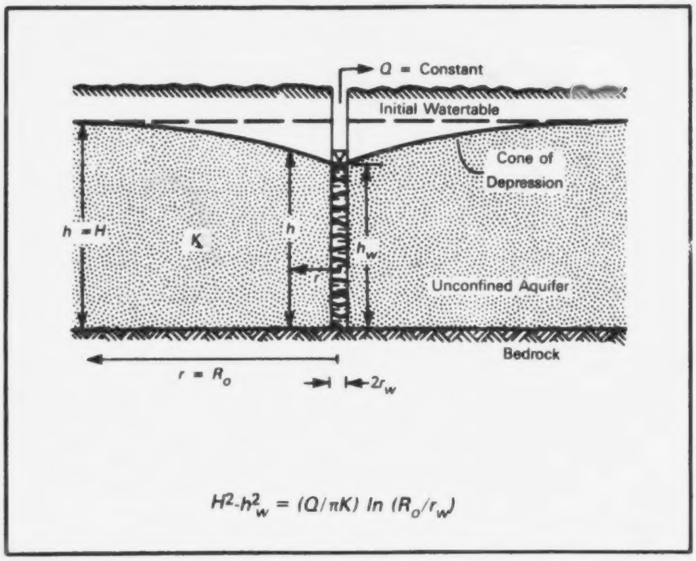
Q = pumping rate (ft^3/day)

K = hydraulic conductivity (ft/day)

R_o = radius of influence of the cone of depression (ft)

r_w = radius of the well (ft).

FIGURE 5-5.
UNCONFINED AQUIFER FLOW

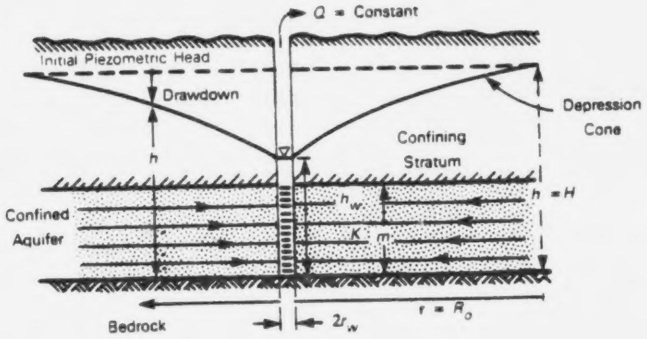


Source: Davis and DeWiest, 1966

Equations for well pumping a confined aquifer (Figure 5-6) are simpler than those for an unconfined aquifer. Drawdown (H-h) at any point r from the well in a confined aquifer can be obtained by:

$$H-h = (Q/2\pi T) \ln (R_o/r_w) \tag{5-5}$$

FIGURE 5-6.
CONFINED AQUIFER FLOW



$$H-h = (Q/2\pi T) \ln (R_o/r_w)$$

Source: Davis and DeWiest, 1966

Plots of H-h versus r on semilogarithmic paper can be developed for both unconfined and confined conditions so that predictions of R_o and wetted screen length can be made. Figure 5-7 shows two such plots.

Nonequilibrium well formulas have an advantage over equilibrium well formulas in that they take into account the effects of varying rates of pumping on well yield and they can be used to predict drawdown at a specified distance from a well at any time after pumping has started. Nonequilibrium well formula are presented in Johnson Division, UOP, Inc. (1975) and USEPA, (1985).

Neither the equilibrium nor nonequilibrium well formula account for the effect of pumping from a well which only partly penetrates an aquifer, cumulative drawdown from a number of wells, or hydrogeologic boundary conditions. Each is discussed briefly below. Specific formulas which account for these effects can be found in Todd (1980), USEPA, (1985), and Johnson Division, UOP, Inc. (1975).

a. Partial Penetration

When a well does not fully penetrate an aquifer, the water close to the well must move along curved lines to reach the well. In doing so, the water must take paths that are longer than strictly radial flow lines (see Figure 5-8). The result of the longer flow paths is that drawdown is greater for a well with partial penetration and, for a given drawdown, the yield of a well partially penetrating the aquifer is less than that of well which fully penetrates the aquifer. Evaluation of the effects of partial penetration can be complicated. However, the effect of partial penetration is negligible on the flow pattern and drawdown beyond a radial distance larger than 0.5 to 2 times the saturated thickness, depending upon penetration (Todd, 1980).

A major problem associated with the use of partially penetrating wells for the containment of plumes is that contaminants can flow under the well if the well does not fully penetrate the aquifer and pumping is not sufficient. Partially penetrating wells may be most useful in collecting floating contaminants.

b. Cumulative Drawdown

In designing a well system for containment or removal of contaminated groundwater, it is frequently necessary to use several wells which are spaced such that their cones of depression overlap and no flow occurs between the wells. It should be noted that overlapping the cones of depression may not be sufficient to capture the plume in aquifers with high natural flow velocities (Keely and Tsang, 1983).

FIGURE 5-7.
PLOTS OF H-h VERSUS r FOR UNCONFINED AND CONFINED AQUIFERS

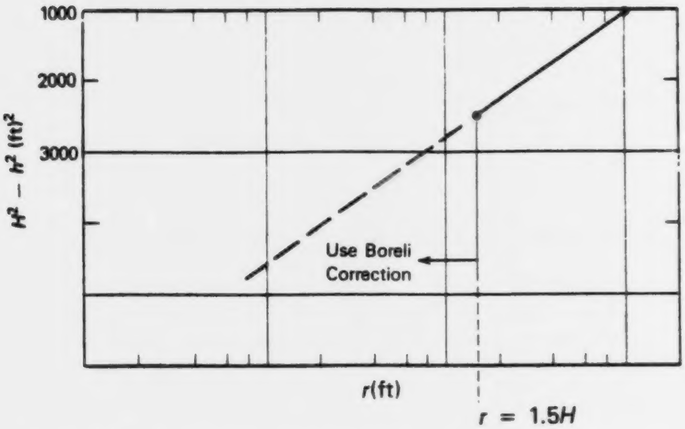


Figure 5-6a. Equilibrium plot for a water table aquifer. $Q = 500$ gpm. $R_o = 1000$ ft. $K = 300$ gpd/ft.^{2} $H = 100$ ft. $r_w = 0.5$ ft.

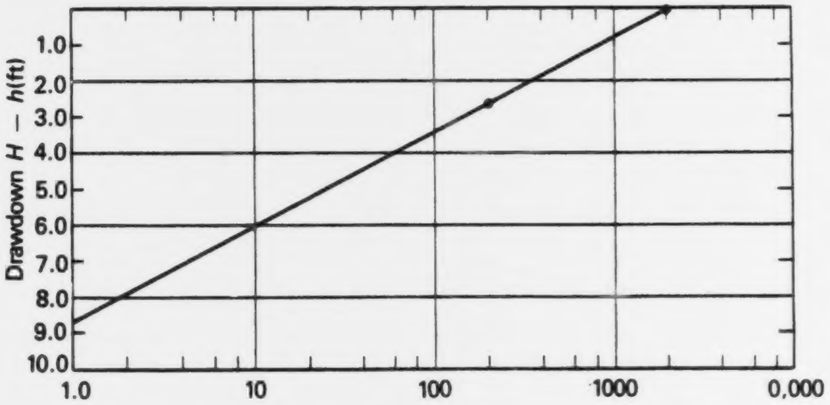


Figure 5-6b. Equilibrium plot for a confined aquifer. $Q = 500$ gpm. $K = 500$ gpd/ft.^{2} $m = 100$ ft. $R_o = 2000$ft. $r_w = 0.5$ ft.

Source: Powers, 1981

FIGURE 5-8.
COMPARISON OF RADIAL FLOW TO A FULLY PENETRATING WELL WITH FLOW TO PARTIALLY PENETRATING WELL

Figure 5-8a. A Radial Flow to a Fully Penetrating Well, Steady State Conditions

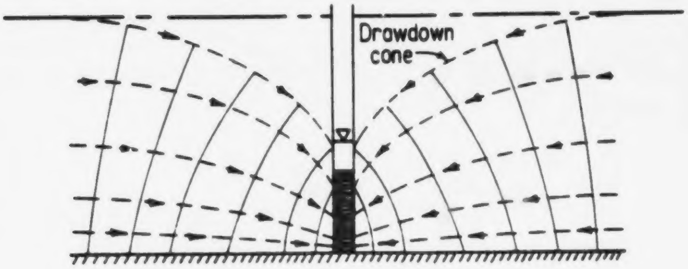
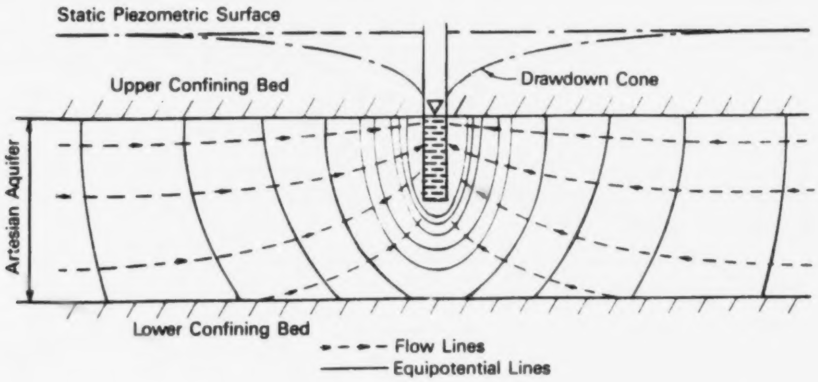


Figure 5-8b. Penetrating 50% of Aquifer



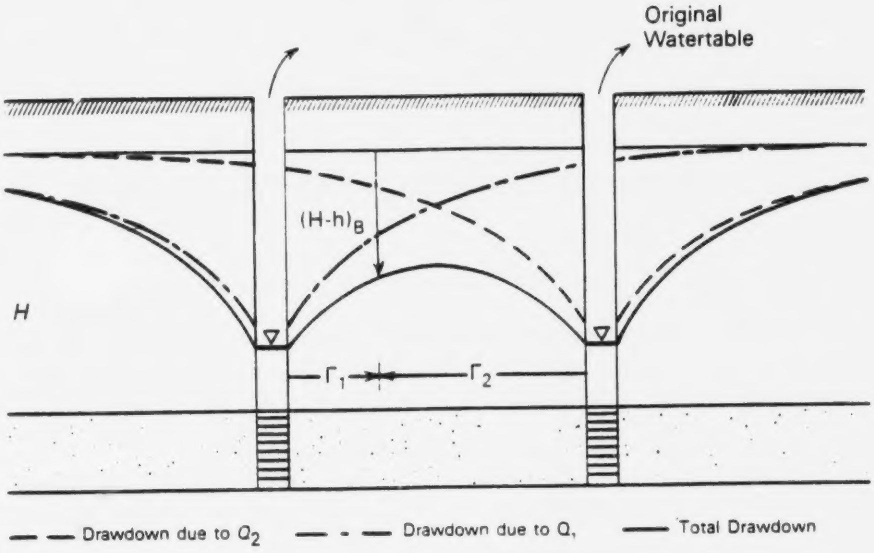
Source: Bureau of Reclamation, 1977

For confined aquifers with several wells tapping the same system, the composite pumping cone can be calculated by summing the individual drawdowns caused by each well (Figure 5-9). This can be accomplished by graphical superposition (i.e., adding the drawdown curves, H-h, for each well) or by calculations using either equilibrium (equation 5-5) or nonequilibrium equations. Theoretically, the method of cumulative drawdown cannot be applied to unconfined aquifers because transmissivity changes with drawdown.

Composite drawdowns for unconfined aquifers under equilibrium conditions can be calculated from:

$$H^2-h^2 = (Q_1 \ln(R_o/r_1)/\ K) + (Q_2 \ln(R_o/r_2)/\ K) + \ldots + (Q_n \ln(R_o/r_n)/\ K) \quad (5\text{-}6)$$

FIGURE 5-9. COMPOSITE DRAWDOWN IN A CONFINED AQUIFER



Source: Freeze and Cherry, 1979

c. Extraction/Injection Well Combinations

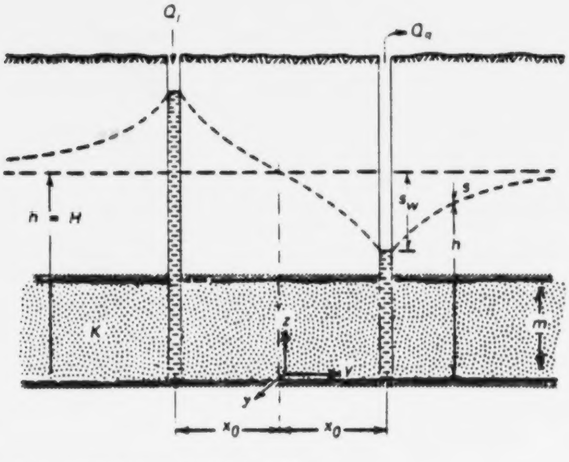
As described in Section 5.1.2, a combination of extraction and injection wells is frequently used in plume management. Injection well theory is identical to extraction well theory except that cones of depression and drawdowns are inverted to above the water table or piezometric surface (cone of impression) (Figure 5-10).

d. Hydrogeologic Boundary Effects

Typically, wells are not located in aquifers that have infinite areal extent, therefore, drawdown cones extend until they intercept a recharge boundary or a barrier boundary. Recharge boundaries (e.g., streams) are areas where aquifers are replenished with water and barrier boundaries (e.g., impermeable zones) are where aquifers terminate. Total drawdown and the rate of drawdown are less than theoretical predictions where recharge boundaries are present and are greater when barrier boundaries are present.

In order to predict head drawdown for the boundary conditions, the methods of images can be used. With this approach, the real bounded system is

FIGURE 5-10.
RECHARGE AND DISCHARGE WELLS IN CONFINED AQUIFER



Source: Davis and DeWiest, 1966

replaced for purposes of analysis by an imaginary system of infinite areal extent (Figure 5-11).

In this system, there are two wells pumping: the real well on the left and an image well on the right. The image well pumps at a rate "Q" equal to the real well and is located at an equal distance "a" from the boundary. If the two component drawdowns in the infinite system are summed it becomes clear that this pumping geometry creates an imaginary impermeable boundary (i.e., a boundary across which there is no flow) in the infinite system at the exact position of the real impermeable boundary in the bounded system.

5.1.3.2 Well Design

Well system design includes a determination of the number of wells needed, the patterns and spacing of the wells, the design of the individual wells, the pumping cycles and rates needed, and the method of handling discharges. The following sections provide some general guidelines for well system design.

Prior to designing the well system, a complete hydrogeologic understanding of the site must be established. A potentiometric surface map (i.e., a map depicting contours of equal head) and a geologic cross-section of the site should be developed. Coefficients of transmissivity (T) and storage (S), discharge (Q), and drawdowns ($H-h_w$) should be determined using pump tests.

a. Equilibrium vs. Nonequilibrium Pumping

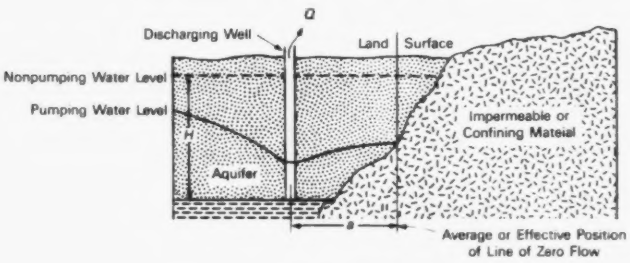
A choice has to be made whether equilibirum or nonequilibrium pumping is to be used at the site because this affects the extent of the radii of influence. Equilibrium pumping will probably be used in most plume management systems, but nonequilibrium pumping does have advantages in some cases. Equilibrium pumping has the advantage of allowing greater well spacing, thereby reducing the number of wells. It is also easier to design than nonequilibirum pumping and may result in lower O&M costs. Nonequilibrium pumping may be a better alternative for aquifers with low hydraulic conductivity, for nonmiscible plumes, and for sites with groundwater barriers or scant recharge.

b. Radius of Influence

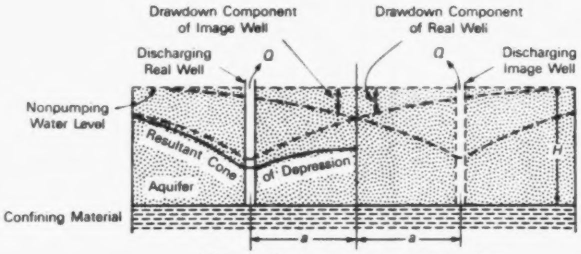
Determining the radius of influence for a well in a given aquifer is critical in remedial action design because it can be used for determining well spacing, pumping rates, pumping cycles, and screen lengths. The radius of influence of a well increases as pumping continues until equilibrium conditions are reached (i.e., when aquifer recharge equals the pumping rate or the discharge rate).

FIGURE 5-11.
METHOD OF IMAGES FOR DETERMINING RESULTANT CONE OF DEPRESSION

Figure 5-13a. Hydraulic Barrier


Discharging Well
Q
Land Surface
Nonpumping Water Level
Pumping Water Level
H
Impermeable or Confining Mateial
Aquifer
a
Average or Effective Position of Line of Zero Flow

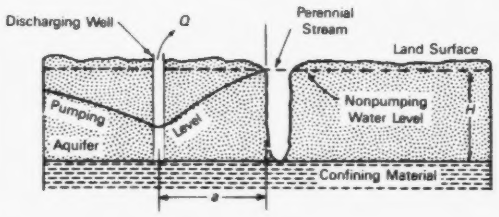
REAL SYSTEM



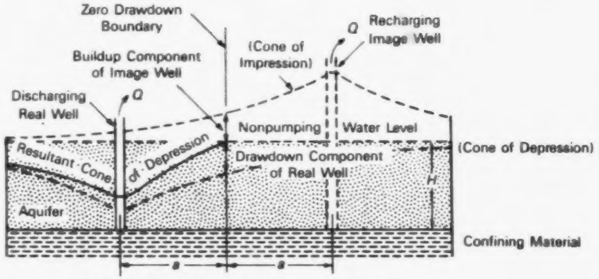
NOTE: Aquifer thickness *H* should be very large compared to resultant drawdown near real well.

HYDRAULIC COUNTERPART OF REAL SYSTEM

Figure 5-13b. Recharge Barrier



REAL SYSTEM



NOTE: Aquifer thickness *H* should be very large compared to resultant drawdown near real well

HYDRAULIC COUNTERPART OF REAL SYSTEM

Source: Ferris et al., 1962, as cited by Lohman, 1972

The most accurate method for estimating the radius of influence is by pumping test analysis. Pumping tests can identify recharge boundaries, barrier boundaries, and slow storage release conditions. The pumping test should be performed until equilibrium conditions are reached. Typical test durations for a confined aquifer are about 24 hours while they may be several days for an unconfined aquifer. Once equilibrium conditions have been reached, the radius of influence for equilibrium or nonequilibrium conditions can be estimated using the equations in Table 5-2.

For an exact solution of the radius of influence under nonequilibrium conditions, a distance drawdown diagram can be constructed using pump test data. Distance drawdown diagrams are plotted on semilogarithmic paper with distance being plotted on the logarithmic scale. Two such diagrams are shown in Figure 5-12. The radius of influence can be obtained from the intercept of the plots with the distance axis at zero drawdown. For cases where pumping rates are variable but all other parameters are held constant, (5-12a) the radius of influence obtained is almost identical. When pumping times are varied (5-12b), the radius of influence is changed such that as pumping time increases, the radius of influence increases. This relationship holds true until equilibrium is reached after which the radius of influence does not increase with continued pumping.

When pumping test data are lacking or incomplete, rough approximations of the radius of influence can be obtained using equations presented in Table 5-2 and the values of transmissivity or hydraulic conductivity, pumping times, and coefficient of storage. Coefficients of storage values typically range from 0.01 to 0.35 for water table aquifers and from 0.00001 to 0.001 for confined aquifers. Typical values are 0.2 for water table aquifers and 0.001 for confined aquifers. Because these estimates are approximate and do not take recharge into account, it is advisable that the value of R_o be adjusted downward so that there will be a greater overlap of the cones of depression and therefore a lower probability that contaminants escape between the wells.

c. Well Spacing

Determination of the proper spacing of wells to completely capture a groundwater plume is probably the most important item in system design. Field practitioners have long had a standing "rule of thumb" for estimating well spacing: adjacent cones of depression should overlap (i.e., radii of influence should overlap). This method is reasonably accurate for aquifers that have low natural flow velocities but will not be valid for aquifers with high natural flow velocities. For these latter cases (and preferably for all cases), velocity distribution plots should be developed to determine well spacing and ensure capture of the plume (Kelly and Tsang, 1983).

Velocity distribution plots for determining well spacing are discussed in USEPA, (1985) and Kelly and Tsang (1983). The use of these plots is based on a determination of the "stagnation point," i.e., the distance downgradient from the well at which the pull of water towards the well by pumping is exactly countered by the natural flow velocity of water away from

TABLE 5-2.
RADIUS OF INFLUENCE EQUATIONS

Pumping Condition	Water Table Aquifer (Unconfined)	Confined Aquifer
Equilibrium		
- Exact	$\ln R_o = K(H^2 - h_w^2)/(458Q)) + \ln r_w$	$\ln R_o = (T(H-h_w)/229Q) + \ln r_w$
Non-equilibrium		
- Exact	Drawdown vs. log distance plots or Theis Method	
Approximate	$R_o = r_w + (Tt/4790S)^{0.5}$	
	$R_o = 3(H-h_w)(0.47K)^{0.5}$	

R_o = Radius of influence (ft)
K = Hydraulic conductivity (gpd/ft^2)
H = Total head (ft)
h_w = Head in well (f)
Q = Pumping rate (gpm)
r_w = Well radius (ft)
T = Transmissivity (gpd/ft)
t = Time (min)
S = Storage coefficient (dimensionless)

Source: USEPA, 1985

FIGURE 5-12.
DISTANCE DRAWDOWN DIAGRAMS FOR A) VARYING PUMPING RATING AND B) VARYING PUMPING TIMES

Figure 12a.

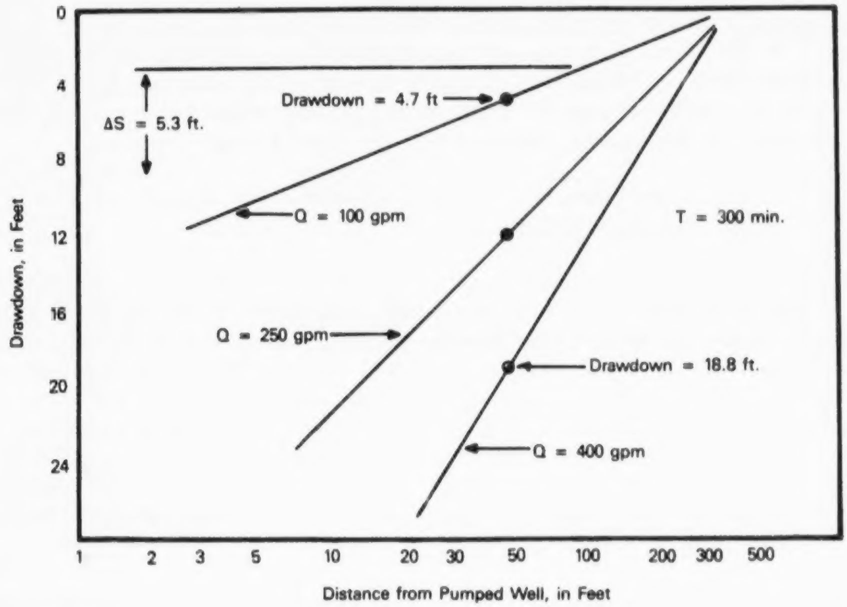
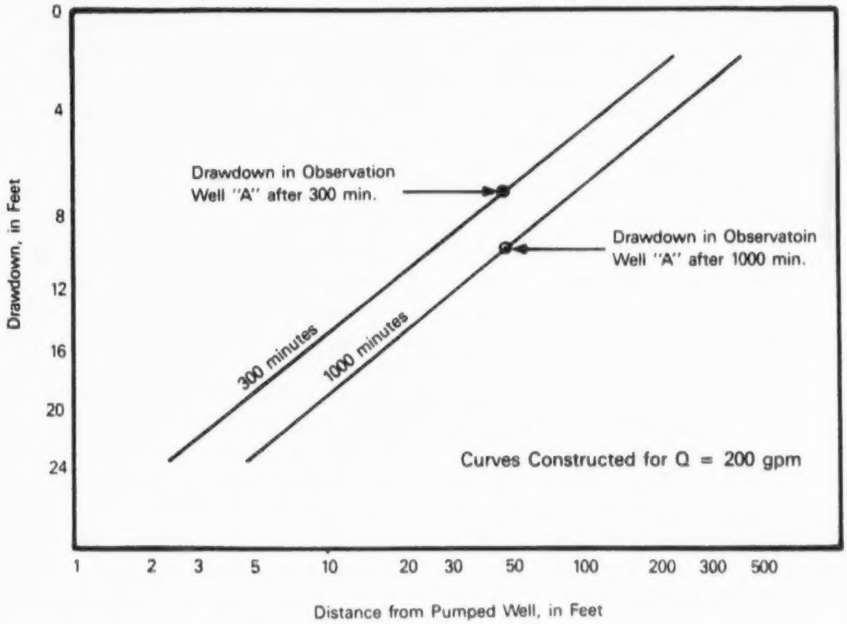


Figure 12b.



Source: Johnson Division, UOP Inc., 1975 the well. The stagnation point is directly related to the pumping rate of the well (i.e., the higher the pumping rate, the further downgradient the stagnation point) and inversely related to the natural flow velocity (i.e., the greater the natural flow velocity, the closer the stagnation point is to the well). Only for the extremely rare case of zero natural flow are the areal boundaries of the capture zone identical to the calculated cone of depression. This means that even though the cones of depression of two pumping wells intersect, they may not be completely capturing the plume unless their capture zones intersect. This finding has significant ramifications for well system design for plume removal (Keely and Tsang, 1983).

d. Pumping Rates

For a confined aquifer, capacity and therefore pumping rate is directly proportional to the drawdown [Q/(H-h)] as long as the aquifer is not unwatered. Increasing the pumping rate will not affect the radius of influence but will affect the amount of time pumping is necessary. Therefore, pumping rates can be selected to suit the situation. In situations where the contaminated plume floats, drawdowns and pumping rates will probably be small. Large drawdowns and high pumping rates are desirable where contaminants are dispersed throughout the aquifer, quick removal is desired, and natural groundwater flow rates are large.

For an unconfined aquifer it has been found that maximum efficiency for well operation occurs at about 67 percent of the maximum drawdown; pumping rates should be adjusted accordingly. This is because part of the formation within the cone of depression is actually unwatered during pumping and the specific yield decreases with increased drawdown. Optimum operating conditions are achieved when the product of specific yield and capacity are greatest and this occurs at about 67 percent of maximum drawdown.

e. System Integration

Once the well spacing, pumping rate, and drawdown have been determined, the system can be designed as a unit. At this point, a decision must be made on the pattern and type (i.e., injection or extraction) of wells to be installed. Numerous patterns of extraction wells or injection wells or both are available. The choice is typically based on whether the design is for containment or removal, the time available for recovery, and the amount of dewatering that is allowable. Patterns that combine extraction and injection wells allow for more rapid contaminant removal without greatly affecting groundwater levels. These patterns are also advantageous because the treated water extracted can be reinjected.

After a well pattern is chosen, the number of wells needed to control the plume must be determined. This is based on the estimated well spacing and the drawdown required. In spacing wells for plume control, it is necessary to have the well's capture zones intersect each other so that contaminants will not flow between wells and escape as described previously.

The number of wells needed can be determined by plotting the chosen pattern of wells with their required spacing on the potentiometric surface map of the site. After this is done, the drawdowns within the radii of influence should be plotted and cumulative drawdowns determined. This will result in a new potentiometric surface map of the site that can be used to identify dead spots where contaminants can escape.

5.1.3.3 Design and Selection of Well Components

As mentioned previously, there are four types of wells used in groundwater pumping: deep wells, ejector wells, wellpoints, and suction wells. This section briefly describes the major components of these well types.

a. Deep Wells

The major components of a deep well are illustrated in Figure 5-13 and include: casing, screen, filter pack and seal, and pump.

Casing--Casing size (diameter) is chosen to satisfy two requirements. The casing must be large enough to accommodate the pump with enough clearance for effective operation and to ensure proper hydraulic efficiency of the well at the intake section (Johnson Division, UOP, Inc., 1975).

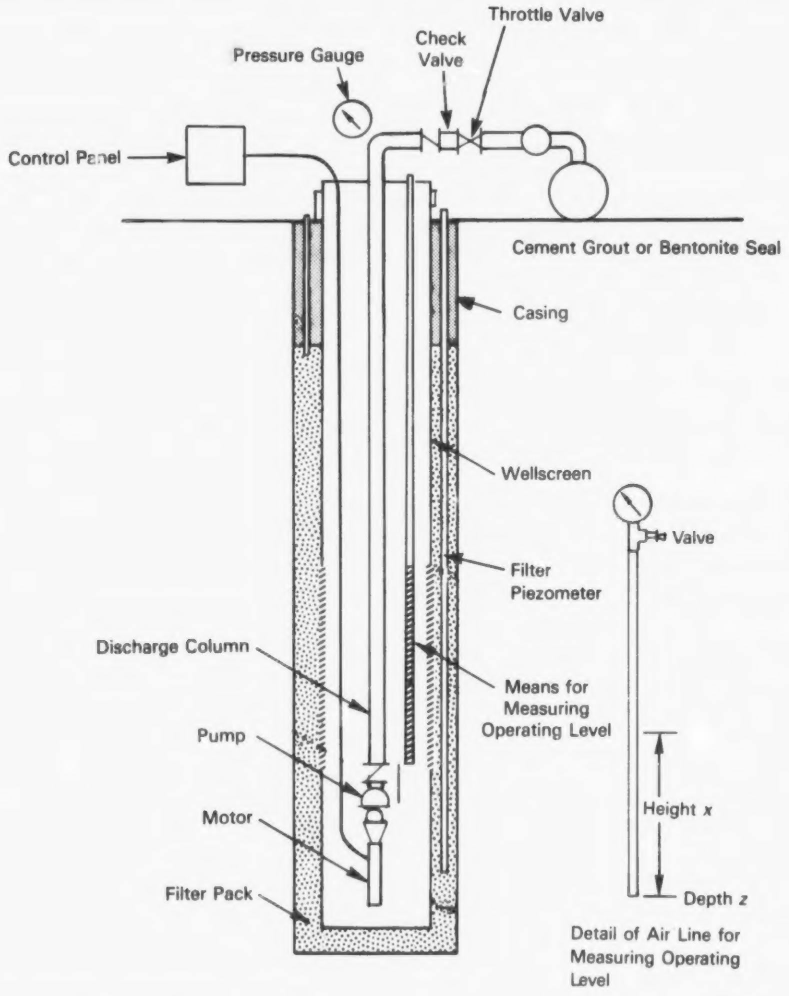
The controlling factor is usually the size of the pump selected for the desired capacity and head. A good rule of thumb is that the casing size should be two standard pipe sizes larger than the nominal diameter of the pump or pump bowl, and not less than one size larger.

Screen--The function of the screen is to stabilize the hole, keep particles out of the well, and facilitate flow into and within the well (Bureau of Reclamation, 1977).

The major variables which need to be considered in selecting screens include:

- Screen type - Several types are available including punched, stamped, louvered, wire-wound perforated pipe, and continuous-slot wire-wound screens.
- Material - Screens are made in a variety of metals, metal alloys, or plastics. Tendency towards corrosion and encrustation are two major factors affecting materials selection. Corrosion of the screen causes enlargement of openings, which could permit excessive sand entrance and premature pump wear. If corrosion is severe, collapse could occur. Encrustation is caused by mineral deposits on the screen

FIGURE 5-13.
COMPONENTS OF TYPICAL DEEP WELL



Source: Adapted from Powers, 1981

surface which tend to plug the openings of the screen and formation. Strength to resist column load and collapse should also be considered in materials selection. Column load results from the screen supporting the weight of the casing. These forces can be considerable in deep wells. Collapse pressure results from earth pressure and caving materials squeezing the screen.

- Screen length - The optimum length of a screen is based on the type and thickness of the aquifer, the available drawdown, and the stratification of the aquifer. Criteria for determining well screen length can be found in Johnson Division, UOP, Inc. (1975) and USEPA (1985).

- Total open area and slot size - A well screen must have a total area of openings such that the entrance velocity does not exceed some critical value which will result in excessive friction losses in the screen openings. Also the slot size must be such that it allows fine grained materials to enter the well and be removed during development but allow coarser material to be retained outside forming a permeable envelope around the well. Procedures for determining the required open area and slot size can be found in Powers (1981) and USEPA (1985).

Gravel Pack--Most wells used for remedial action will probably be drilled, which normally requires that the area around the well screen be filled with a filter or gravel. This filter performs several important functions (Powers, 1981):

- Fills the annular space preventing the uncontrolled collapse of the formation against the screen

- Retains a sufficient percentage of fines thus preventing them from being pumped continuously

- Passes some amount of fines and mud cake that have built up on the sides of the hole

- Transmits water freely from the aquifer to the screen during pumping.

To perform these functions, the filter should be a very uniform material so that it has a high permeability and can be placed without segregation. The filter should be as coarse as possible without continuously passing fines (Powers, 1981). Various formulas for relating gravel pack grain size gradations to aquifer grain size gradations have been developed. Procedures can be found in Powers (1981) and USEPA (1985).

Pumps--The turbine submersible pump and the vertical line shaft pump are generally used for deep wells. Submersible pumps have the advantage of being relatively slender for their capacity allowing their use in small diameter wells. Commercially available pumps can be obtained with capacities greater than 100 gallons per minute and with motors of several hundred horsepower.

Vertical lineshaft pumps are similar to submersible pumps except that the motor is located on the surface rather than being attached directly to the pump. This type of pump is used for extracting high volumes of groundwater and is available in sizes ranging from a few horsepower to over a thousand horsepower. The cost of a lineshaft pump is typically greater than for a submersible pump of similar horsepower size.

Before selecting a pump for any installation, it is necessary that accurate information be available with regard to total capacity, operating conditions (e.g., pumping cycle, peak load, future needs), and total head. The total dynamic head (h_t) of a pump represents the total vertical lift of water from the well.

Total dynamic head (h_t) can be calculated using the equation:

$$h_t = h_e + h_f + h_v \qquad (5\text{-}7)$$

where:

h_e = total vertical lift, from the pumping level in the well to the water discharge point (ft)

h_f = total frictional losses in pipes and fittings expressed as head (ft)

h_v = velocity head required to produce the desired flow (ft).

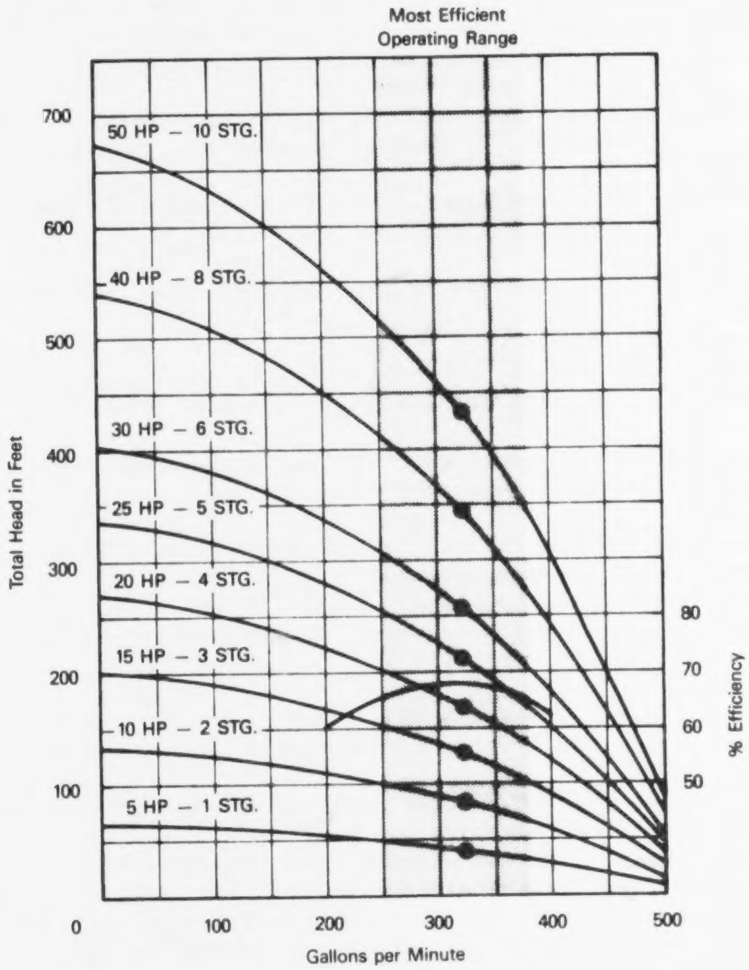
Total vertical lift (h_e) is the sum of the vertical distance from the pumping level in the well to the outlet and the hydrostatic pressure head at the discharge outlet. Frictional losses in pipes and fittings are calculated using tabulated values found in Johnson Division, UOP, Inc. (1975) and USEPA (1985). Velocity head (h_v) is determined using the formula $h_v = 0.155V^2$, where V is the velocity of flow (ft/sec) and the coefficient 0.155 has units of sec^2/ft. Usually, h_v is very small and is only taken into account where very high discharges are expected at very low lift and discharge head, or where velocity of flow is very high.

Once the total dynamic head is obtained and the capacity is determined, a pump and motor can be chosen using performance curves available from manufacturers. These curves typically plot capacity versus total head and percent efficiency of the system. An example of performance curves for 325 gallon per minute pumps is given in Figure 5-14.

b. Ejector Wells

Ejector wells have certain advantages over wellpoints and deep wells because they are not typically depth limited as wellpoints are and they are less expensive than deep wells when close spacing is required. The biggest drawback to the use of ejector wells is that they are very inefficient (typically less than 15 percent efficiency). Ejector wells can be used

FIGURE 5-14.
PERFORMANCE CURVE



Source: Flint and Walling Inc.

independently of each other or arranged so that they utilize a common pumping system.

The typical components of a two-pipe and a single-pipe ejector well system are illustrated in Figure 5-15. The lift principle for the two-pipe model is (Powers, 1981):

- High pressure supply water (Q_1) moves down the supply pipe through ports in the ejector body to the tapered nozzle where the pressure head is converted to water velocity
- Supply water exits the nozzle at less than atmospheric pressure creating a vacuum in the suction chamber
- Groundwater (Q_2) is drawn into the chamber through the foot valve because of the pressure differential
- Supply water and groundwater (Q_1 + Q_2) are mixed in the suction chamber
- The mixed water enters the venturi where the velocity decreases because of divergence resulting in increased pressure
- The increase in pressure develops sufficient head to return the combined flow to the surface.

The lift principle for the single-pipe model, illustrated in Figure 5-15, is similar except that the supply water under pressure (Q_1) flows downward between the well casing and the inner ejector return pipe, and a packer assembly separates the supply water from the groundwater so that different pressures are developed (Powers, 1981).

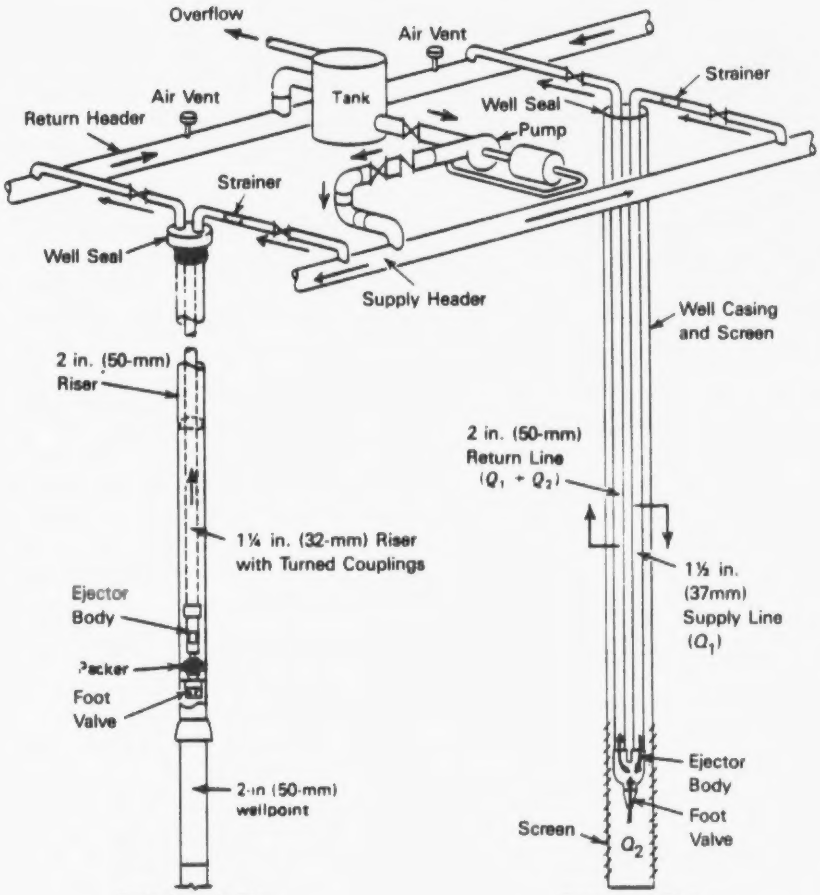
Single-pipe ejector wells are most commonly used because they require less piping and yields are greater for smaller diameter casings.

Ejector pumps consist of a water tank and a pump with the required valves and piping. Each well can have its own pump or one pump can be used by a number of wells. Water from the storage tank is pumped under pressure through a header pipe that supplies the ejector wells. Return flow, a mixture of supply water and groundwater, recharges the tank through the header system. Excess water in the tank is discharged to a treatment system.

The proper ejector pump and ejectors (i.e., nozzle and venturi) can be selected using manufacturer's data or through calculations. The procedures for calculating the size of an ejector are to calculate head ratios, estimate the capacity ratios, and calculate the diameters of the nozzle and the venturi. The procedure is fairly complex and is described in detail by Powers (1981).

Materials for risers, swings, headers, tanks, screens, and pumps should be selected to minimize corrosion and encrustation. Selection criteria are

FIGURE 5-15.
COMPONENTS OF ONE-PIPE AND TWO-PIPE EJECTOR WELLS



Typical Single-Pipe Ejector Installed in a 2-in. (50-mm) Wellpoint

Typical Two-Pipe Ejector Installed in a 6-in. (150-mm) Well

Source: Powers, 1981

the same as for deep well systems. However, ejectors must also be protected to maintain well performance. Ejectors used in corrosive environments can be made of plastic to eliminate the problem. Ejector sensitivity to clogging should be a primary consideration in selection.

c. Wellpoints

Wellpoint systems consist of a group of closely spaced wells connected to a header pipe and pumped by a suction pump. Wellpoints are best suited for groundwater extraction in stratified soils where total lift or drawdown will not exceed 22 feet. The advantages to using wellpoints are that the system design is flexible and the wellpoints are relatively inexpensive even when closely spaced.

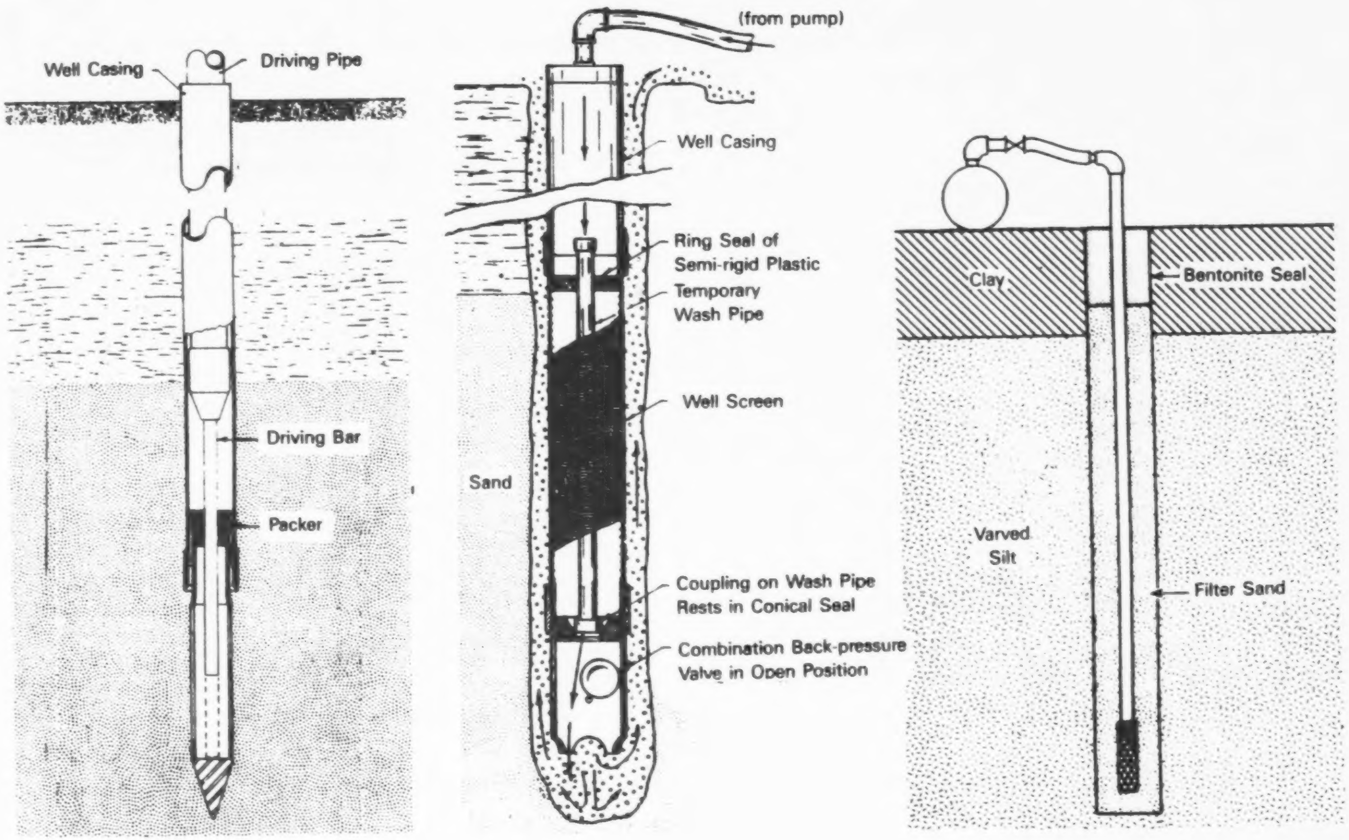
A suction (vacuum) pump is typically used in wellpoint systems to lift water. Suction pumps (either oil sealed or water sealed) accomplish lift by developing a negative pressure head at the pump intake rather than by applying force to the water source as in ejector pump systems. The maximum lift attainable by suction pump is about 22 feet.

Wellpoints are specially made well screens that are typically 1.5 to 3.5 inches in diameter and are capable of yields up to 35 gallons per minute. Large wellpoints (up to 8 inches in diameter) generally called suction wells, are also available. They have capacities greater than 35 gallons per minute. Figure 5-16 shows the basic types of wellpoints. Wellpoint screens can be made of heavy wire mesh, continuous wire, slotted plastic, or perforated plates. As with deep wells, the materials selected should minimize the potential for corrrosion and encrustation.

Because wellpoints are typically installed in oversized boreholes, filter sands are placed around the wellpoint to fill the annular space. The filter sands increase the effective diameter of the wellpoint, decrease the entrance velocities of water, prevent clogging of the screen with fines, and provide for vertical drainage from overlying layers (Powers, 1981).

The depth at which wellpoints are set is dependent upon the hydrogeology of the site. Although drawdown is limited to approximately 22 feet below land surface, wellpoints can be set at almost any depth depending on the situation. Where contaminants float at the top of the aquifer, wellpoints can be set at shallow depths. Where contaminants sink or mix with the groundwater, the wellpoints can be set deeper. The only criterion for wellpoint depth is to avoid dewatering below the screen. If this occurs, air enters the system and reduces the vacuum and therefore the drawdown. During normal operation of the system, dewatering below the top of a wellscreen can sometimes occur. The problem can be minimized in the field by adjusting the valves that control individual wells.

FIGURE 5-16.
DRIVEN WELLPOINT (a), JETTED WELLPOINT (b), AND DRILLED WELLPOINT (c)



Source: Johnson Division, UOP Inc., 1975 a and b; Powers, 1981c

5.1.4 Installation of Wells

Well installation typically consists of four steps:

- Opening the borehole by dislodging and removing earth materials
- Installing a casing which may or may not be installed simultaneously with opening the borehole
- Completing the well by installing screens, filters, pumps, and grout
- Developing the well which consists of dislodging and removing fines that have built up on the well screen, the filter, and the walls of the aquifer during the previous steps.

Each of these steps is described briefly in this section.

5.1.4.1 Opening the Borehole

Methods for opening boreholes are selected based on well depth and subsurface geologic and hydrologic characteristics. Shallow wells (<100 feet) are augered, driven, or jetted. For deep wells (≥100 feet), rotary drilling, jetting, and cable-tooling are generally used. Table 5-3 briefly describes the applications of these methods to various geologic and hydrologic conditions. Screens and casings may be installed simultaneously with the opening of the borehole or after the borehole has been completed, depending upon the technique used to open the borehole. Casing and screen installation methods are also summarized in Table 5-3.

a. Hand Augering

Hand augers are available in several shapes and sizes, all operating with cutting blades at the bottom that bore into the ground with a rotary motion. When the blades are full of loose earth, the auger is removed from the hole and emptied; the operation is repeated until the desired hole depth is reached (Todd, 1980).

b. Driven Wells

A driven well consists of a series of connected lengths of pipe driven by repeated impacts into the ground to below the water table. Water enters the well through a drive (or sand) point at the lower end of the well. This consists of a screened cylindrical section protected during driving by a steel cone at the bottom. Driving can be done with a maul sledge, drop hammer, or air hammer (Todd, 1980).

TABLE 5-3.
METHODS OF WELL INSTALLATION

Method		Applications			Casing/Screen Treatment	Comments
Basic	Variations	Geologic Material	Max. Well Dia. (in)	Max. Well Depth (ft)		
Hand	Augers	Soft, without excess sand and water, no boulders	6	20	Driven after opened	
	Driving Points	Soft soil free of boulders	3	30	Driven as hole proceeds	Wider and deeper holes can be machine driven
Boring	Rotary Auger Bucket	Soft soils without excess boulders	48	90	Driven after hole opened	Typically limited to 24-in diameter wells or less
	Spiral Auger	Soft soils without excess sand and water, no boulders	6	90	Driven after hole opened	Can not be used below water table, must be used in combination with other techniques
Jetting	Self jetting	Soft soils free of boulders	8	50	Drives as hole opened	Jetted wells require less development
	Wellpoint/Riser Unit	Soft soils free of boulders	8	50	Driven as hole opened	Method uses large quantities of water
	Separate Temporary Jetting Pipe	Soft soils free of boulders	8	100	Placed after hole opened	
	Separate Permanent Jetting Pipe	Soft soils free of boulders	8	100	Driven as hole opened	

TABLE 5-3.
METHODS OF INSTALLATION (CONTINUED).

Method		Applications			Casing/Screen Treatment	Comments
Basic	Variations	Geologic Material	Max. Well Dia. (in)	Max. Well Depth (ft)		
Jetting (cont.)	Rotary Hole-puncher	Soft soil, sandstone, schist	24	200	Driven as hole opened	
Rotary Drilling	Conventional Hydraulic	Any type	--	--	Placed after hole opened	Typically used for deep wells, quickest drilling method
	Reverse Hydraulic	Any type, boulders may be a problem	60	--	Placed after hole opened	Requires large quantities of water
	Air Rotary	Any type	12	--	Placed after hole opened	Very fast method
	Air Rotary with Pneumatic Hammer	Any type	8	--	Placed after hole opened	
Cable Tool	Conventional	Any type	--	--	Driven during or after hole opened	Typically used for deep wells, provide ability to closely monitor and log well
	California Stove Pipe	Any type	--	--	Pushed with jacks as hole opened	Quicker and less expensive than conventional cable tool

Source: USEPA, 1985

c. Rotary Bucket Augering

The bucket auger consists of a cylindrical steel bucket with a cutting edge projecting from a slot in the bottom. The bucket is filled by rotating it in the hole by a drive shaft of adjustable length. When full, the auger is hoisted to the surface and the excavated materials are removed through hinged openings on the side or bottom of the bucket. Reamers, attached to the top of the bucket, can enlarge holes to diameters exceeding the auger size (Todd, 1980).

d. Spiral Augers

There are two types of spiral augers: Solid-stem and hollow-stem continuous flight augers.

Solid-stem augers consist of auger flights welded to a solid core. Drilling is accomplished by rotation of the augers which convey soil samples to the surface.

A hollow-stem auger consists of flights welded to a hollow core which has a 1-1/2- to 6-1/4-inch inside diameter. The drilling technique is the same as for the solid-stem augers except that the core facilitates downhole sampling and well installation.

e. Jetting Methods

Jetted wells are constructed by the cutting action of a downward-directed stream of water. The high velocity stream washes the earth away, while the casing, which is lowered into the deepening hole, conducts the water and cuttings up and out of the well.

Jetting methods for installing small diameter, shallow wells include the following:

- Self-jetting, permanent drop tube--The entire wellpoint is jetted down and remains in place. The permanent drop tube eliminates the need for temporary jetting pipes. Riser tubes are used to extend the length of the well as jetting proceeds and act as casings.

- Self-jetting, separate drop tube--Same as the self-jetting, permanant drop tube method except that the drop tube is removed after jetting is completed.

- Separate, temporary jetting pipe--A capped pipe with cutting teeth at the bottom is jetted down by water forced through openings in the cap. If soil becomes difficult to remove, the pipe is rotated back and forth to aid the jetting process. Once the well is pushed to the final depth, the wellpoint and riser are placed in the jetting pipe and the jetting pipe is removed.

- Separate, permanent jetting pipes--Similar to the separate, temporary method except that the wellpoint with screen is packed into the jetting pipe, the pipe is lifted to expose the screen, and the jetting pipe then becomes the riser casing.

f. Rotary Drilling

Rotary drilling methods involve the use of a rotating bit and a viscous fluid or air to transport the cuttings. Four rotary drilling methods are commonly used (Luhdorff and Scalmanini, 1982; Johnson Division, UOP, Inc., 1975):

- Conventional Hydraulic (direct circulation)--This practice employs a drilling fluid which is circulated down a rotating drill pipe through the drill bit. The fluid returns up the annulus of the borehole, removing the drill cuttings. The mixture goes to a settling pit, and the fluid is again recirculated through the fluid system by a pump. Hole stability is accomplished by the hydrostatic pressure of the drilling fluid.

- Reverse Hydraulic (reverse circulation)--In this procedure, the flow of the drilling fluid is reversed, allowing the circulation of fluid from the bit up the rotating drill pipe to a settling pit. This permits holes of larger diameters to be constructed. The integrity of the borehole is achieved by hydrostatic fluid pressures created by maintaining the hole full of water during drilling operations. The procedure is generally cheaper, and reduces the need for specialized drilling mud control and development time required for normal mud rotary well construction. However, there is the potential for loose, permeable soils to cave in.

- Air Rotary--This method uses compressed air as the drilling fluid instead of water or drilling mud. A high, uphole velocity of air is used to remove cuttings from the borehole. This method is good for consolidated rock drilling and will allow faster penetration and longer bit life as long as water infiltration into the hole is small.

- Air Rotary with Pneumatic Hammer--Drilling equipment capable of using either air or drilling mud as the fluid is modified to include tophead drive, casing hammer operations. Such a rig provides the ability to alter the method of construction to meet varying hole conditions encountered in the well. The use of a casing hammer allows casing to be installed through difficult drilling formations such as unconsolidated surface deposits and then returns to either air or mud circulation drilling for hole completion.

g. Cable Tool

The cable tool or percussion drilling method involves the raising and lowering of a string of drilling tools suspended on a drilling line in the well bore, followed by bailing the drilled cuttings from the hole. Generally, the well bore is kept open by the installation of a casing string as the drilling operation proceeds to the completion depth. Cable tooling is much slower than rotary drilling methods.

Conventional cable-tool equipment consists of a bit, a stem for length and weight, jars to loosen stuck stem and bits, and a rope socket which connects the entire string to the drill cable.

The California stove pipe is a modification of the conventional cable tool. It has the same basic operating principle as the conventional cable tool. However hydraulic jacks are used to force the casing downward rather than driving the casing by the impact of tools; a mud scow is used as both a drill bit and a bailer, and a thin pipe within a pipe is used as casing as opposed to standard line pipe (Johnson Division, UOP, Inc., 1975).

5.1.4.2 Well Completion

Once the borehole has been opened, installation of a screen filter pack and grout is necessary to complete the well prior to well development. The method of completion is different for gravel pack and natural wells.

a. Filter Pack Wells

The method typically used to complete a well with an artificial filter is the double-casing method. In this method, a string of outside casing, corresponding to the size of the outside diameter of the filter pack (i.e., the borehole), is installed as the hole is drilled or after it is completely opened. A second string of casing containing the well screen is then centered within the outer casing. The selected filter material is then placed between the inner and outer casings and the outer casing is pulled back. The outer casing may be removed completely or left in place above the level of the screen.

The top of the annular space above the filter is sealed with grout (e.g., cement, clay) as is the space between the outer casing and the aquifer. The top of the inner casing is sealed with a lead slip packer. Pumps are then installed into the inner casing and the well is developed.

Jetted wellpoints are completed in a manner similar to drilled wells. In this case, filter sands are packed around the wellpoint and grout is installed from the top of the filter to the surface. The grout prevents surface water infiltration into the well and minimizes the chances of air entering the wellpoint.

b. Naturally Developed Wells

In consolidated formations, where the material surrounding the well is stable, groundwater can enter directly into an uncased well. For these naturally developed wells, grouting and sealing are done prior to installing the screen. If a temporary outside casing has been installed during drilling, the casing must be removed while the grout is still fluid. This allows for a good seal between the borehole walls and the grout. Once the grout has set, the plug can be drilled out and the well screen installed.

Numerous methods are available for installing well screens depending on the type of well screen used, the drilling equipment, the geologic material, and the presence of grout. Once the well screen is installed, the pump can be placed within the casing. Well development can then take place to ensure adequate yield.

5.1.4.3 Well Development

Well development is the process where fine soil materials are removed from in and around the screen, allowing water to flow freely. This process is accomplished through one or several methods of surging water or air through the well screen and into or out of the surrounding material. The well development process:

- Removes materials that have built up in the openings of the screen during the well drilling and installation processes
- Removes fines from the sides of the borehole that resulted from the drilling procedure, e.g., drilling mud
- Increases the hydraulic conductivity of adjacent geologic materials and the filter pack by removing fine materials
- Stabilizes the fine materials that remain in the vicinity of the well and retards their movement into the well.

The benefits of well development are increased yields, reduced pumping of fines which can damage pumps, and decreased corrosion and encrustation.

5.1.5 Maintenance and Performance Monitoring

Pumps, casing, and screens must be maintained to ensure a constant reliable flow of water from the well. Proper well maintenance is especially important in plume management because the loss of a well could result in contaminant escape. The causes of well yield loss and failure are: encrustation, corrosion, and pump failure which is typically caused by sand intrusion, wear on mechanical parts, or electrical failure.

Prior to beginning any well maintenance, a preliminary evaluation must be conducted to identify whether or not the problem can be corrected. Operational records will determine the normal operating conditions of the well and aid in evaluating the problem. Removing the pumps and checking the casing and screens may be necessary. This can also be accomplished by downhole video equipment.

General maintenance procedures that can be used for encrustation, corrosion, and related pump problems include: chemical treatment of casings, screens, and pumps; mechanical and electrical maintenance of pumps and well developments. Chemical treatment involves the use of acids, biocides, and phosphates to dissolve deposits. Acids are used to dissolve inorganic substances which have formed (e.g., calcium, magnesium, and iron deposits). Phosphates act as dispersing agents to help break up clays, colloids, and some metal deposits.

The performance of a pumping system should be monitored using observation wells that are sampled periodically for contaminants. The location of these wells can be determined from the potentiometric surface maps. Observation wells should be located to monitor deadspots and areas where cones of depression overlap.

5.1.6 Technology Selection/Evaluation

Groundwater pumping systems are the most versatile and flexible of the groundwater control technologies. As mentioned previously they can be used to contain, remove, or divert a plume under a wide variety of geologic and hydrogeologic conditions (confined and unconfined aquifer; consolidated and unconsolidated materials to any depth; and homogeneous and heterogeneous aquifers). When used together with a barrier wall and a cap, complete hydrologic isolation of a site can be achieved. Groundwater pumping systems, however, perform poorly in low transmissivity aquifers.

Operational flexibility is high since pumping rates can be modified to adjust to changes in flow rate. System performance is generally good provided the wells are properly designed and maintained. Deadspots and areas where cones of depression overlap should be continuously monitored to ensure effectiveness. The reliability of pumping systems can be adversely affected by mechanical and electrical failure of pump which can result in loss of contaminants. However, repairs and replacement of parts can be done quickly and easily.

Well systems are generally safer to install than drains and barrier walls since there is no need for trench excavation. Installation is relatively easy and quick. Contractors qualified to drill and install wells are readily available.

One of the biggest drawbacks with pumping systems is that operation and maintenance costs are high, especially when used as a long term remedial action.

5.1.7 Costs of Well Systems

Costs of well systems for plume management can vary greatly from site to site. Some of the factors that determine these costs are the geology, the characteristics of the contaminated and naturally occurring groundwater, the extent of contamination, the periods and duration of pumping, local wage rates, the availability of supplies and equipment, and the electrical power required. Costs associated with a well system can be categorized as mobilization costs, installation and removal costs, and operation and maintenance costs.

Mobilization costs include all costs incurred in obtaining equipment and having it available at the site. Some of the items included in mobilization costs are (Powers, 1981):

- Installation equipment including drilling rigs, jetting equipment, and well development equipment
- Pumping equipment including pumps, contacts, hoses, and cables
- Standby equipment including generators, switches, pipes, and cables
- Equipment rental, repair, delivery, and handling costs
- Utility installation
- Enclosures for storing equipment
- Engineering and geotechnical services including the design of the system, submittal preparation, field testing, and on-site supervision during installation
- Waste, water, and soil treatment including transport, treatment, and disposal
- Decontamination of drill rigs and tools
- Health and safety precautions
- Lodging and per diem for drilling personnel.

Installation and removal costs include the costs for crews and equipment necessary to install the well system at the site. These costs should also include allowances for setup, cleanup, foul weather, and other miscellaneous delays that typically occur. These costs are extremely dependent on the conditions under which the well or system must be installed. Costs are generally obtained for drilling on a per foot or time and material basis. Some reasons for variations in well drilling and installation costs are:

- Well diameter
- Well depth
- Well components including well screens, casing, well pumps, motors, controls, discharge columns, well heads, fittings, collector pipes, and power lines
- Drilling specifications (e.g., double cased versus single cased)
- Geologic material being drilled
- Health and safety requirements
- Sampling requirements
- Site access.

Because of the above listed variables, the cost of well installation can vary considerably from site to site. To accurately estimate costs associated with the installation of a well system, as much information as is available should be obtained and evaluated prior to system design.

Operation and maintenance costs are typically high for pumping systems. In some cases, these costs can be greater than the initial installation and mobilization costs.

Removal costs will probably be incurred at all sites at some point when pumping is no longer required. Removal costs can be offset somewhat by the salvage value of the removed equipment. However, decontamination of the equipment may be more costly than the salvage value.

The above list of cost related items is not all inclusive; some sites will require additional items. If long-term operations are expected, operation and maintenance and removal costs should include escalation factors. Tables 5-4 through 5-6 give some typical costs that may be incurred for the items mentioned above. These costs are presented as ranges and variation can be expected depending on the complexity of well specification and system designs. Before costs can be accurately determined, detailed information is required.

TABLE 5-4
1984 COSTS FOR SELECTED PUMPS AND ACCESSORIES
(STANDARD PUMPS AND JACUZZI)

Pump/Accessory	Description	Cost Range
Jet Pump		
- shallow well	pumping depths: 25 ft horsepowers: 1/3 to 1-1/2 hp capacities: 60 to 27000 gph	$200-500
- deep well	pumping depths: 320 ft horsepowers: 1/3 to 2 hp capacities: 60 to 1000 gph	$250-650
- jets and valves	single pipe jets	$40-100
	double pipe jets	$30-75
- seals	single or double pipe	$15-40
- foot valves		$10-50
- air volume controls		$10-30
Submersible Pumps		
- 4-inch pump	pumping depths: 900 ft horsepowers: 1/3 to 3 hp capacities: 50 to 2000 gph	$425-1500
- control boxes		$75-150
- magnetic starters		$160-250
- check valves		$15-420
- well seals		$20-120
Vacuum Pumps		
- diesel motors	800-7000 gpm (48,000-420,000 gph)	$13,000-50,000
- electric motors	800-7000 gpm (48,000-420,000 gph)	$9,000-35,000

Source: USEPA, 1985

TABLE 5-5

1985 COSTS FOR WELLSCREENS AND WELLPOINTS

(Johnson Division, UPO, Inc., and Gator Plastics Inc.)

Type	Description	Costs
Drive Wellpoints	stainless steel, 1-1/4 to 2-inch diameter	$34-43/ft
	low carbon steel, 1-1/4 to 2-inch diameter	$16-30/ft
	PVC plastic, 1-1/4 to 2-inch diameter	$5-6/ft
Wellscreens	stainless steel, 1-1/4 to 36-inch diameter	$33-540/ft
	low carbon steel, 1-1/4 to 36-inch diameter	$18-157/ft
	PVC plastic, 1-1/4 to 12-inch diameter	$10-60/ft
Jetting Screens (fittings)	cast iron or mild steel, 2 to 12-inch diameter	$30-270/ft

TABLE 5-6
1985 DRILLING COSTS FOR UNCONSOLIDATED MATERIALS AND INSTALLING[1] 2- TO 4-INCH DIAMETER WELLS

Drilling Method	Average Production Rates (ft/hr)	Range of Drilling Costs
Conventional Hydraulic Rotary	40-50	$25-40/ft
Reverse Circulation Hydraulic Roters	40-50	$35-45/ft
Air Rotary	50-60	$17-25/ft
Auger (Hollow Stem)	20-40	$11-22/ft
Bucket Auger	40-50	$10-20/ft
Cable Tool	3-5	$15-17/ft
Hole Puncher (Jetting)[2]		$40/ft
Self Jetting[2]		$22/ft
Mobilization		$500-600/rig

[1] Includes drilling, well material, and installation costs.

[2] Includes rental of all necessary equipment, e.g., well points, pumps, and headers.

Source: (1) Blas, S. Empire Soils Investigations, Inc., Groton, NY. Personal communication, October-November 1984.

(2) Baker, P. P.C. Exploration, Inc., Roseville, CA. Personal communication, October-November 1984.

(3) Craddock, D.F. Stang Drilling & Exploration, Rancho Cordova, CA. Written communication, May 15, 1981.

The complexity of developing costs for groundwater pumping systems led Geraghty and Miller, Inc. to develop a method for estimating total capital and operating costs for deep wells based on the use of existing hydraulic models (Lundy and Mahan, 1982). They have applied the model to a number of scenarios and the resulting cost estimates provide considerable insight into how aquifer characteristics affect cost and how the cost of well system components compares to total capital costs.

Table 5-7 summarizes seven recovery system cost scenarios to which the cost methodology was applied. For these scenarios, the plumes were assumed to be moving in unidirectional flow fields and to have the following dimensions:

- 250 to 2500 feet wide
- 500 to 5000 feet long
- 25 to 250 feet deep.

A high transmissivity (100,000 gal./day/ft.) is assigned to four low-flux scenarios. A low transmissivity (5000 gal./day/ft.) is assigned to three high flux scenarios.

TABLE 5-7. SUMMARY OF SEVEN RECOVERY SYSTEMS COST SCENARIOS ($1985)*

AQUIFER AND PLUME CHARACTERISTICS DESIGN PARAMETERS		DELINEATION (K)	DESIGN (K)	WELLS/DRAINS (K)	SURFACE INFRASTRUCTURE (K)	TREATMENT FACILITY (K)	WELLS/DRAINS (O&M)	TREATMENT (O&M)	MONITORING (O&M)	TOTAL K	TOTAL O&M
Low Flux, High Transmissivity (100,000 gal/day/ft.) (plume width x length x depth, ft)											
– (250 x 500 x 25)	2 wells; 2gpm	83	28-110	16.5	39	33	16.5	5.5	11	198-281	33
– (250 x 500 x 250)	2 wells; 2gpm	165	28-110	55	39	33	22	5.5	11	320-402	39
– (2500 x 5000 x 25)	2 wells; 20gpm	220	28-110	16.5	165	44	16.5	5.5	11	474-567	33
– (2500 x 5000 x 250)	2 wells; 20gpm	441	28-110	66	165	44	22	5.5	11	744-827	39
High Flux, Low Transmissivity (5000 gal/day/ft.)											
– (250 x 500 x 25)	4 wells; 40gpm	83	28-110	33	39	55	16.5	16.5	11	237-320	44
– (250 x 500 x 250)	4 wells; 40gpm	165	28-110	121	39	66	22	16.5	11	408-490	50
– (2500 x 5000 x 250)	4 wells; 400gpm	441	28-110	143	165	121	50	55	11	900-981	116

*Cost in thousands of dollars;
Costs were updated to 1985 costs using the
Engineering News Record Construction Cost Indices for 1982 and 1985.

Source: Modified from Lundy and Mahan, 1982

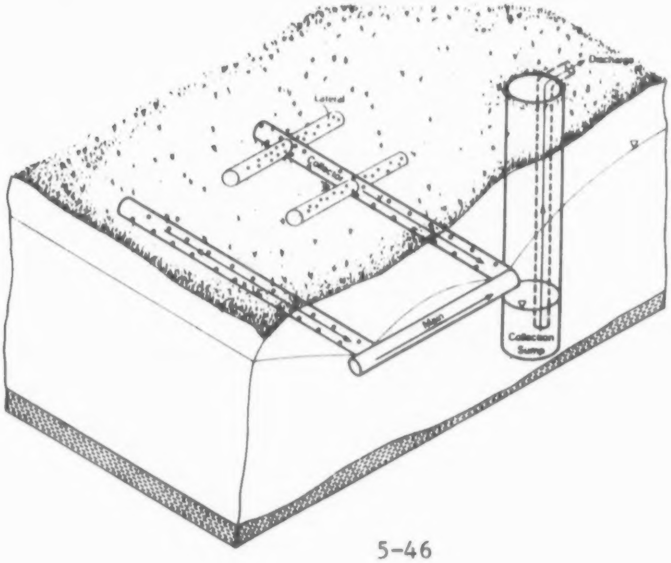
As table 5-7 illustrates, the total capital costs range from $198,540 to $981,670 or $982 to $3,970/ft. O&M costs range from $33,090 to 115,815/year. These costs probably represent the low end of costs encountered in the field because of the more complex hydrogeological conditions frequently encountered. A large percentage of total capital costs are spent on plume delineation and well design. Plume delineation or remedial investigation costs vary from 26 percent of the total capital costs for a small plume wtih contamination depth of about 25 feet to 59 percent of total costs for a large plume with contaminatin to considerable depths. Well design costs range from only about 3 percent up to 39 percent. The capital costs for the well system components ranges from about 6 percent to 30 percent and O&M costs account for 37 to 57 percent of the total O&M costs. Although well system costs are relatively small, the discharge from the wells influences infrastructures and treatment costs. In a few of the scenarios presented, these two cost elements are significant and account for up to 44 percent of the capital cost and 48 percent of the O&M costs.

5.2 Subsurface Drains

5.2.1 Description

Subsurface drains include any type of buried conduit used to convey and collect aqueous discharges by gravity flow. Subsurface drains essentially function like an infinite line of extraction wells. They create a continuous zone of influence in which groundwater within this zone flows towards the drain. Subsurface drainage components are illustrated in Figure 5-17.

FIGURE 5-17.
SUBSURFACE DRAINAGE SYSTEM COMPONENTS



The major components of a subsurface drainage system are:

- Drain pipe or gravel bed--for conveying flow to a storage tank or wet well. Pipe drains are used most frequently at hazardous waste sites. Gravel bed or french drains and tile drains are used to a more limited extent.
- Envelope--for conveying flow from the aquifer to the drain pipe or bed
- Filter--for preventing fine particles from clogging the system, if necessary
- Backfill--to bring the drain to grade and prevent ponding
- Manholes or wet wells--to collect flow and pump the discharge to a treatment plant.

5.2.2 Applications/Limitations

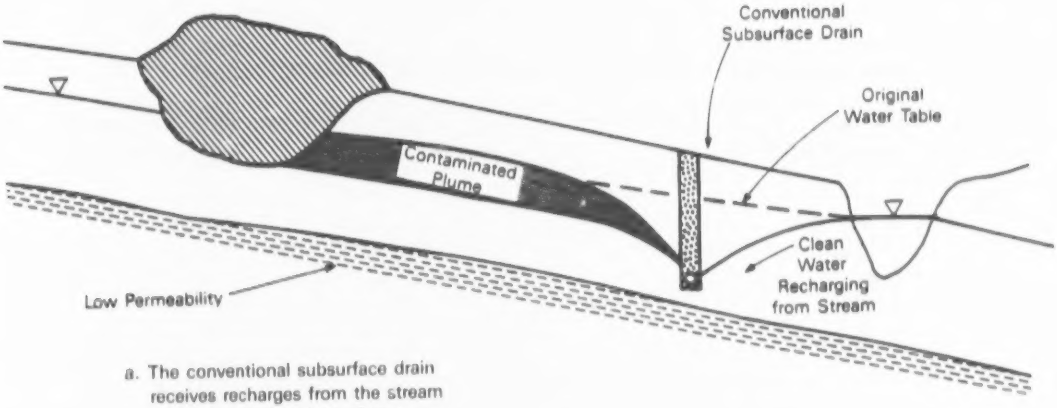
Since drains essentially function like an infinite line of extraction wells, they can perform many of the same functions as wells. They can be used to contain or remove a plume, or to lower the groundwater table to prevent contact of water with the waste material. The decision to use drains or pumping is generally based on a cost-effectiveness analysis.

For shallow contamination problems, drains can be more cost-effective than pumping, particularly in strata with low or variable hydraulic conductivity. Under these conditions, it would be difficult to design and it would be cost-prohibitive to operate a pumping system to maintain a continuous hydraulic boundary. Subsurface drains may also be preferred over pumping where groundwater removal is required over a period of several years, because the operation and maintenance costs associated with pumping are substantially higher.

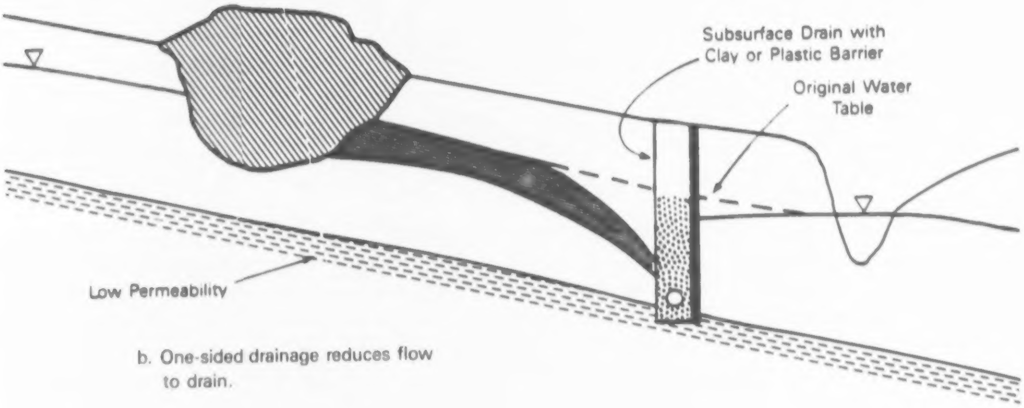
One of the biggest drawbacks to the use of subsurface drains is that they are generally limited to shallow depths. Although it is technically feasible to excavate a trench to almost any depth, the costs of shoring, dewatering, and hard rock excavation can make drains cost-prohibitive at depths of less than 40 feet. However, in stable low permeability soils where little or no rock excavation is required, drains may be cost-effective to depths of 100 feet.

The most widespread use of subsurface drains at hazardous waste sites is to intercept a plume hydraulically downgradient from its source (Figure 5-18a). Frequently, these interceptor drains, as they are commonly called, are used together with a barrier wall (Figure 5-18b). There are two primary reasons for the interceptor drain/barrier wall combination. In the case where

FIGURE 5-18.
THE USE OF A ONE SIDED SUBSURFACE DRAIN FOR REDUCING FLOW FROM UNCONTAMINATED SOURCES


Conventional Subsurface Drain
Original Water Table
Contaminated Plume
Clean Water Recharging from Stream
Low Permeability

a. The conventional subsurface drain receives recharges from the stream as well as the leachate plume, resulting in larger collection and treatment.



b. One-sided drainage reduces flow to drain.

Source: USEPA, 1985

a subsurface drain is to be placed just upgradient of a stream, the drainge system would reverse the flow direction of the stream and cause a prohibitively large volume of clear water to be collected. The addition of a barrier wall would prevent infiltration of clean water from the stream thereby reducing treatment costs.

In another application, where the primary remedial action involves installation of a downgradient barrier wall to contain wastes, an interceptor drain can be installed just upgradient of the wall to prevent overtopping and to minimize contact with wastes which may degrade the wall.

Subsurface drains can also be placed around the circumference of a waste site in order to lower the groundwater table (Figure 5-19) or to contain a plume. A circumferential subsurface drain may be part of a total containment system which consists of a barrier wall and a cap in addition to the subsurface drain (Figure 5-20).

In addition to depth, other limitations to the use of subsurface drains include the presence of viscous or reactive chemicals which could clog drains and envelope material. Conditions which favor the formation of iron manganese or calcium carbonate deposits may also limit the use of drains.

For hazardous waste site applications, pipe drains are most frequently used. French or gravel drains can be used where the amount of water to be drained is small and flow velocities are low. If used to handle high volumes or rapid flows, these drains are likely to fail due to excessive siltation, particularly in fine grained soils. Tile drains have not been widely used in hazardous waste site applications.

5.2.3 Design

The major elements to consider in designing a subsurface drainage system include:

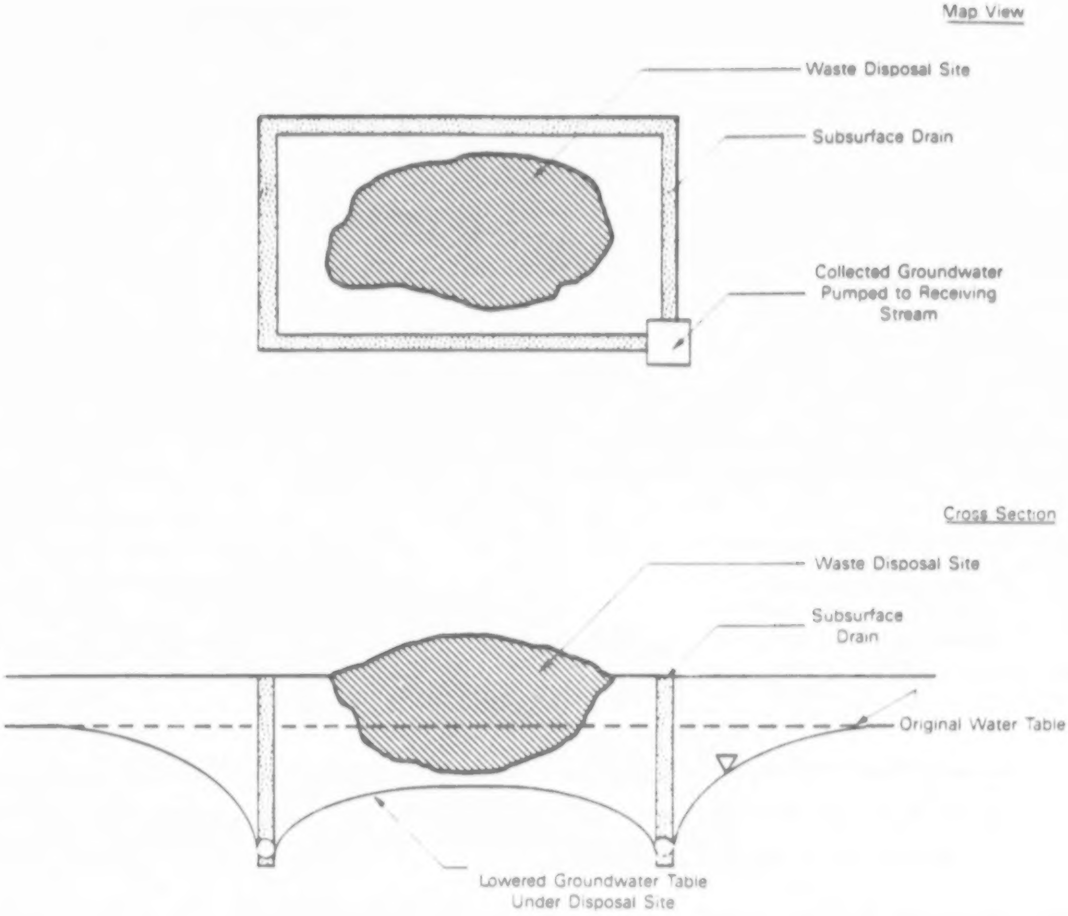
- Location and spacing of drains to achieve desired head levels
- Hydraulic design of the conduit including pipe diameter and gradient
- Properties and design of the envelope and filter materials
- Design of a pumping station.

Each of these design elements is discussed in this section.

5.2.3.1 Location and Spacing of Drains

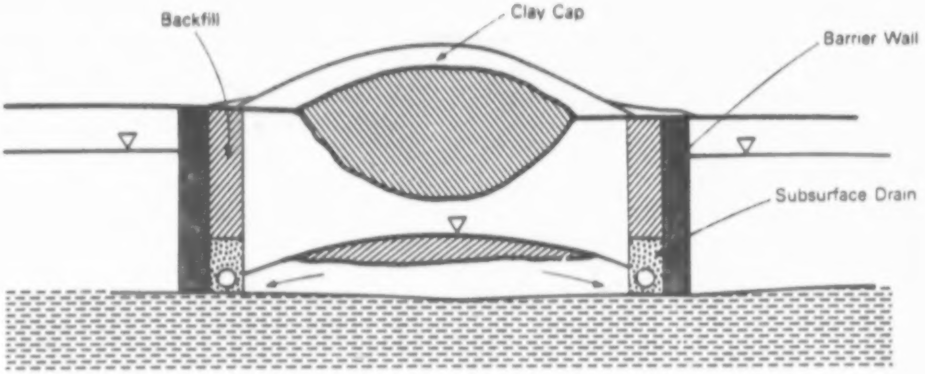
For the purposes of designing subsurface drainage systems, drains have been divided into two categories based on their function: interceptor and

FIGURE 5-19.
THE USE OF SUBSURFACE DRAINAGE TO LOWER GROUNDWATER LEVELS



Source: USEPA, 1985

FIGURE 5-20.
THE USE OF SUBSURFACE DRAINAGE IN A COMPLETELY ENCAPSULATED SITE



Source: USEPA, 1985

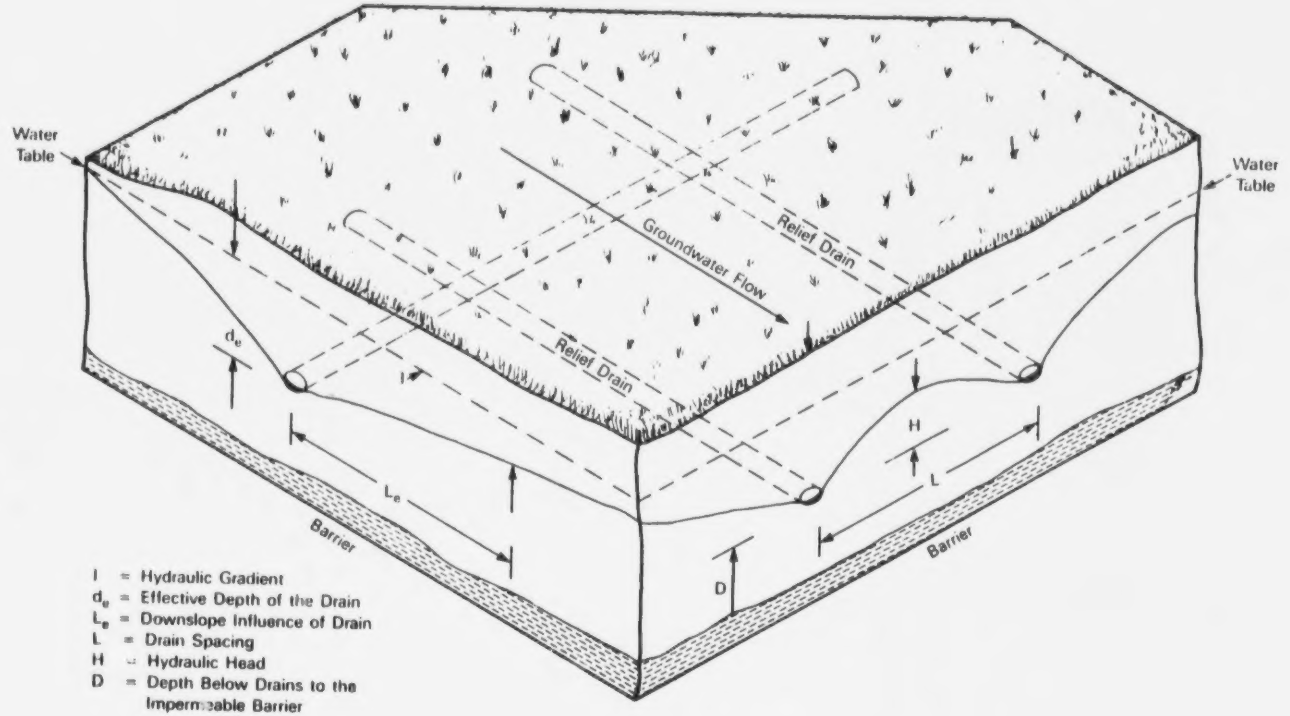
relief drains. Interceptor drains are installed perpendicular to groundwater flow and are used to intercept groundwater from an upgradient source. Relief drains are installed parallel to the direction of flow or around the perimeter of a site where the water table is realtively flat. Relief drains are used primarily to lower the water table beneath a site. Figure 5-21 shows the effect of interceptor and relief drains in altering the configuration of the water table.

a. Locating Interceptor Drains

Determining the required location for an interceptor drain is more often based exclusively on the use of field data than on theoretical design. Remedial investigation data are used to develop potentiometric surface maps, hydraulic conductivity data, plume boundary limits, and geologic cross-sections. With this data in hand, the design engineer can pinpoint and stake the design drain line. Additional borings are then taken along this line and the alignment shifted if needed to obtain proper interception.

To function properly, an interceptor drain should be installed perpendicular to groundwater flow direction. In stratified soils having greatly different hydraulic conductivities, the drain should be installed resting on a layer of low hydraulic conductivity. If the trench is cut through an impervious stratum, there is danger that a significant percentage of the leachate moving laterally will bridge over the drain and continue downgradient. Similarly, if soil layers or pockets with high hydraulic conductivity underly the drain, the leachate may flow beneath the drain.

FIGURE 5-21.
THE AFFECT OF RELIEF AND INTERCEPTOR DRAINS IN ALTERING THE CONFIGURATION OF THE WATER TABLE



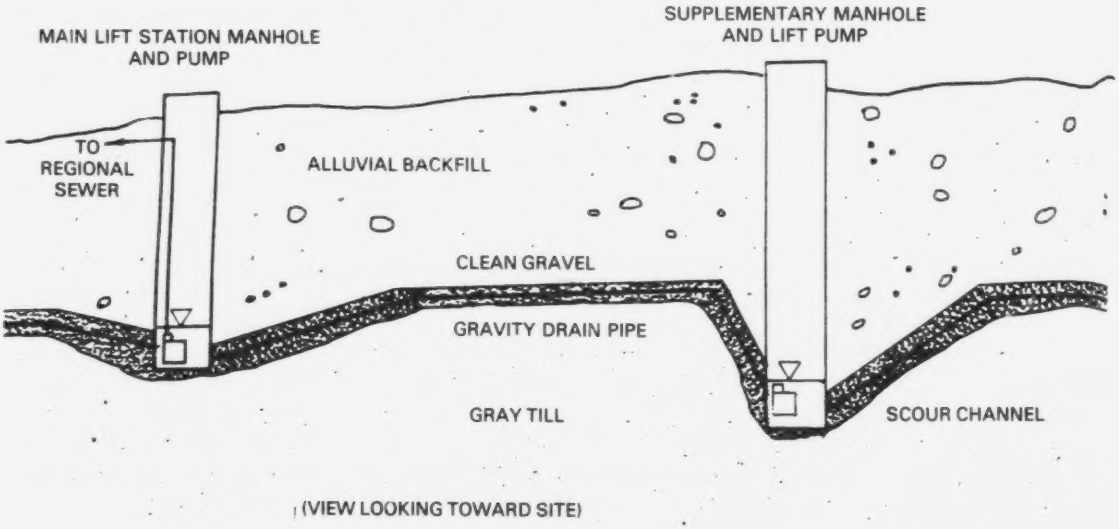
I = Hydraulic Gradient
d_e = Effective Depth of the Drain
L_e = Downslope Influence of Drain
L = Drain Spacing
H = Hydraulic Head
D = Depth Below Drains to the Impermeable Barrier

Source: SCS, 1973

The problem of underflow beneath the drain has been solved at waste sites using various approaches. Underflow can be minimized by placing impermeable liner material at the base of the trench before laying a thick (1 to 3 foot) gravel bedding. Where pockets of highly permeable soils are found (e.g., scour channel in an alluvial area), it is possible to construct a manhole at the lowest point of the permeable soil and to install a small lift station and force main to carry the leachate from this low area back up to the adjacent gravity flow section of the drainage system (see Figure 5-22). A third solution is to install a barrier wall downgradient of the drain and key it in to a low permeability layer.

In order to decide where to position a drain, the design engineer will need a reasonably good estimate of the upgradient and downgradient influence of the drain. In general, the shape of the drawdown curve upgradient of the site is independent of hydraulic conductivity but is a function of head. Therefore, the influence of the drain extends for a distance which is greater the more gradual the water table gradient. Theoretical determination of the

FIGURE 5-22.
SUBSURFACE DRAIN WITH A LIFT STATION


MAIN LIFT STATION MANHOLE AND PUMP
SUPPLEMENTARY MANHOLE AND LIFT PUMP
TO REGIONAL SEWER
ALLUVIAL BACKFILL
CLEAN GRAVEL
GRAVITY DRAIN PIPE
GRAY TILL
SCOUR CHANNEL
(VIEW LOOKING TOWARD SITE)

Source: Giddings, 1982

upgradient influence can be expressed by the following equation developed by Glover and Donnan (1959) and described by Van Hoorn and Van der Molen (1974):

$$D_u = 1.33\ m_s I \tag{5-8}$$

where:

D_u = Effective distance of drawdown upgradient (ft)

m_s = Saturated thickness of the water bearing strata not affected by drainage (ft)

I = hydraulic gradient (dimensionless).

The depth to which the water table is lowered downgradient of the interceptor is proportional to the depth of the drain. Theoretically, a true interceptor drain lowers the water table downgradient to a depth equal to the depth of the drain. The distance downgradient to which the drain is effective in lowering the water table is infinite provided recharge is not occurring. This is never the case since infiltration from precipitation always recharges the groundwater.

In some cases, it is not necessary to know the downgradient influence of the drain. It may be adequate to cut-off the upgradient source of the plume and allow contaminated groundwater downgradient to continue on its course. This will depend on the size of the plume and the use of groundwater downgradient.

The theoretical determination of the downgradient influence can be obtained from the following equation (Figure 5-23):

$$D_d = (KI/q)\ (d_e - h_d - D_2) \tag{5-9}$$

where:

D_d = downgradient influence (ft)

K = hydraulic conductivity (ft/day)

I = hydraulic gradient (dimensionless)

q = drainage coefficient (ft/day)

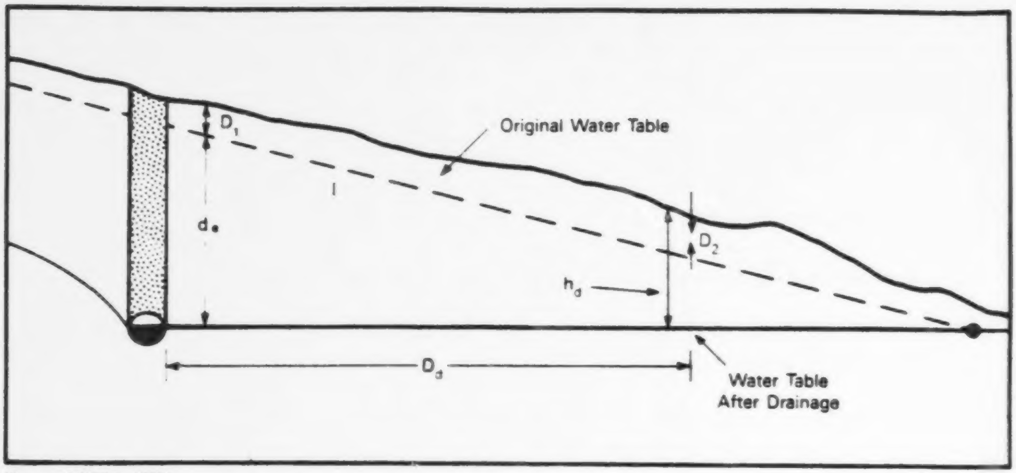
d_e = depth of drain (ft)

h_d = desired depth of drawdown (ft)

D_2 = distance from ground surface to water table prior to drainage at the distance D_d downgradient from the drain (ft)

In the equation given above, D_d and D_2 are interdependent variables. In obtaining the solution to the equation, the value of D_2 is estimated, then a

FIGURE 5-23.
SYMBOLS FOR THE GLOVER AND DONNAN EQUATION FOR CALCULATING THE DOWNGRADIENT INFLUENCE OF AN INTERCEPTOR DRAIN



Source: USEPA, 1985

trial computation is made. If the actual value of D_2 at distance D_d is appreciably different, a second calculation is necessary. Where I is uniform throughout the area, D_2 can be considered equal to D_1 (i.e., the distance from the ground surface to the water table measured at the drain). If a second interceptor is needed to lower the water table to the desired depth, it would be located D_d feet downgradient from the first.

b. Depth and Spacing of Parallel Drains

This section presents basic equilibrium equations for estimating the spacing of drains under two conditions: drains resting on an impermeable layer and drains above an impermeable layer. These equations assume that steady state conditions exist, recharge distribution and leachate generation over the area between the drains is uniform, and the soil is homogeneous. Since real world situations rarely meet these criteria, results obtained using these equations should be considered approximate and a conservative design approach should be taken to ensure that the desired head levels are maintained. Numerous computer models are available for obtaining more exact solutions to drain spacing problems.

Drains on an "Impermeable" Barrier--Drains are often used where the depth to a low permeability barrier is relatively shallow and the drains can be laid just above the barrier. In developing and using drain spacing formulas for this case, an underlying soil layer is considered to be "impermeable" if the hydraulic conductivity is less than one-tenth that of the above soil layer (Wesseling, 1973).

Flow to drains resting on a low permeability layer is illustrated in Figure 5-24. This flow can be represented by an equation developed by Donnan (1946) and described by Wesseling (1973):

$$L = [(8KDH + 4KH^2)/q]^{0.5} \tag{5-10}$$

where:

L = drain spacing (ft)

K = hydraulic conductivity of the drained material (ft/day)

D = distance between the water level in the drain line and the impermeable barrier (ft)

H = water table height above the drain levels at the midpoint between two drains (ft)

q = leachate generation rate (ft/day).

For a pipe drain resting on an impermeable barrier, the parameter D approximately equals the radius of the pipe and hence is very small in comparison to H (the water table height above the drain). This allows equation 5-10 to be simplified to:

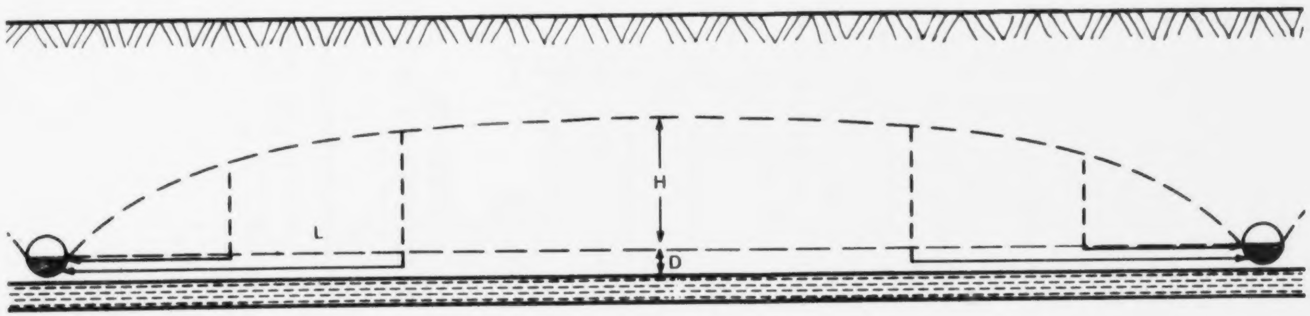
$$L = [(4KH^2)/q]^{0.5} \tag{5-11}$$

As Figure 5-24 illustrates, when two parallel drains are installed, each exerts an influence (L or the drain spacing), which in theory will intersect with each other midway between the drain lines. The influence (L) is the distance from the drain to a point where the drawdown can be considered insignificant and is commonly referred to as the zone of influence.

Drain spacing (L) and hydraulic head level (H) in the equations are interdependent design variables which are a function of the leachate generation rate (q) and hydraulic conductivity (K) of the drained material. Assuming constant leachate generation and hydraulic conductivity, the closer two drains are spaced the more their drawdown curves will overlap and the lower the hydraulic head levels between the drains will be.

Successful application of a subsurface drainage system under these conditions requires that the drains can be placed close enough together to intercept the entire plume or to lower the groundwater table to the level required to prevent contact of the water with the waste material. However, a minimum drain spacing may be imposed by the boundaries of the waste because excavation through the waste material can be extremely hazardous.

FIGURE 5-24.
FLOW TO A DRAIN RESTING ON A LOW PERMEABILITY BARRIER



Source: USEPA, 1985

Drains Above a Low Permeability Barrier--In many instances, it may not be possible to install drains to the depth of a low permeability barrier because the cost of installing drains to this level is prohibitively high or because the plume does not extend to the depth of the barrier. In these instances flow is not adequately described using equations 5-10 and 5-11.

Figure 5-25 illustrates the flow to a drain not resting on a low permeability layer. The flow lines are not parallel and horizontal as shown in Figure 5-24, rather, they converge towards the drains. The convergence, or radial flow as it is commonly called, causes a more than proportional loss of hydraulic head because the flow velocity in the vicinity of the drains is larger than elsewhere in the flow region. The effect is that the elevation of the water table and the drain spacing would be larger than would be predicted using equation 5-10.

Hooghoudt (1940) as described by Wesseling (1973) developed a modified drain spacing formula which accounts for radial flow and head loss. His method accounts for head loss by using an equivalent depth, d, to replace D in equation 5-10. The equation can be used to describe the conditions shown in Figure 5-25, that is, flow to drains which are installed at the interface of a two layered soils with hydraulic conductivities of K_1 and K_2. For this condition, the equation can be written as:

$$L = [(8K_2dH + 4K_1H^2)/q] \qquad (5\text{-}12)$$

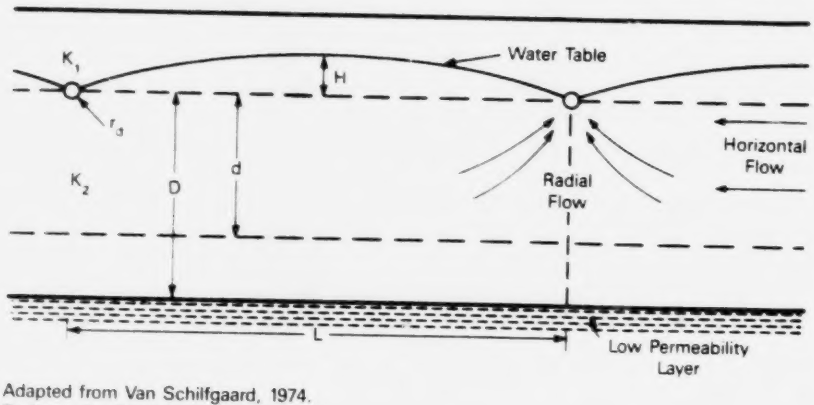
where the new terms are defined as:

d = equivalent depth (ft)

K_1 = hydraulic conductivity of the layer above the drain (ft/day)

K_2 = hydraulic conductivity of the layer below the drain (ft/day).

FIGURE 5-25.
FLOW TO DRAIN NOT RESTING ON A LOW PERMEABILITY BARRIER



Adapted from Van Schilfgaard, 1974.
Drainage for Agriculture, Agronomy
Monograph No. 17. Pages 245-270.

In equation 5-12 both drain spacing L and equivalent depth d are unknowns. The value of d is typically calculated from a specified value for L, so that equation 5-12 cannot be solved explicitly in terms of L. The use of this equation as a drain spacing formula involves either a trial and error procedure of selecting d and L until both sides of the equation are equal or the use of nomographs which have been developed specifically for equivalent depth and drain spacing (see Wesseling, 1973).

For saturated thicknesses (D) greater than 32.8 feet, the equivalent depth can be calculated from drain spacing using the following equation (USEPA, 1985):

$$d = 0.57(L) + 0.845 \qquad (5\text{-}13)$$

Again, it should be noted that the Hooghoudt equation, though widely used in drain spacing, is only accurate when the level of the drain corresponds with the interface between the two soil levels. When drains are being placed so that the interface lies either above or below the drains, drain spacing may not be accurately predicted by this formula. Computer models or equations developed by Ernst (1962) and described by Wesseling (1973) could be used for a more accurate prediction of drain spacing.

5.2.3.2 Pipe Diameter and Gradient

Pipe diameter and gradient are the design parameters used to ensure that water which arrives at the drainline can be conveyed without a build up of pressure. The formula used for hydraulic design of pipes is based on the Manning formula for pipes which is:

$$Q = AR^{0.67}\, I^{\,0.5}/N \qquad (5\text{-}14)$$

where:

A = drainage area (ft^2)

Q = design discharge (ft^3/sec)

R = hydraulic radius (ft) equal to the wetted cross-sectional area A_w divided by the wetted perimeter (1/4 the diameter for full flowing pipes)

I = hydraulic gradient

N = roughness coefficient.

Each of these factors is described further below.

a. Hydraulic Gradient and Roughness Coefficient

In designing subsurface drainage systems, a gradient is chosen which is great enough to result in a flow velocity that prevents siltation (>1.4 feet per second) but will not cause turbulence (critical velocity) (SCS, 1973). Velocities of less than 1.4 feet per second are acceptable if filter fabrics are used to prevent filtration. Critical velocities for various soil types are summarized below (SCS, 1973):

Soil Types	Velocity (ft/sec)
Sand and Sandy Loam	3.5
Silt and Silt Loam	5.0
Silty Clay Loam	6.0
Clay and Clay Loam	7.0
Course Sand and Gravel	9.0

Table 5-8 gives the gradients for different sizes of drains which result in the critical velocity for drains with a roughness coefficient of 0.011, 0.013, and 0.015. The roughness coefficient is a function of the hydraulic resistance of the drain material. It should be obtained from the pipe manufacturer prior to hydraulic design. Commonly used drainage pipes include perforated PVC and flexible corrugated PVC (favored for their low cost and chemical resistance); steel (rugged, more costly, subject to corrosion); aluminum (light weight, easy to handle, subject to some forms of corrosion); and vitrified clay.

b. Design Discharge

Design discharge, Q, is equal to the sum of the individual discharges which impinge upon the drain. Estimates of the total discharge can be obtained using the water balance method. This method provides an estimate of the amount of percolation that will recharge the water table between the lines of the drain. Once the percolation rate has been calculated, discharge can be obtained by multiplying the percolation route by the drainage area:

$$Q = q \times A \tag{5-15}$$

where:

Q = design discharge (ft^3/sec)

q = leachate generation rate (ft/sec)

A = drainage area (ft^2).

TABLE 5-8
DRAIN GRADES FOR SELECTED CRITICAL VELOCITIES

Drain Size (Inches)	Velocity (ft/sec)					
	1.4	3.5	5.0	6.0	7.0	9.0
Grade--feet per 100 feet						
For drains with "N" = 0.011(a)						
Clay Tile, Concrete Tile, and Concrete Pipe (with good alignment)						
4	0.28	1.8	3.6	5.1	7.0	11.5
5	0.21	1.3	2.7	3.9	5.3	8.7
6	0.17	1.0	2.1	3.1	4.1	6.9
8	0.11	0.7	1.4	2.1	2.8	4.6
10	0.08	0.5	1.1	1.5	2.1	3.5
12	0.07	0.4	0.8	1.2	1.6	2.7
For drains with "N" = 0.013						
Clay Tile, Concrete Tile, and Concrete Pipe (with fair alignment)						
4	0.41	2.5	5.2	7.5	10.2	16.8
5	0.31	1.9	3.9	5.6	7.7	12.7
6	0.24	1.5	3.1	4.4	6.0	10.0
8	0.17	1.0	2.1	3.0	4.1	6.8
10	0.12	0.8	1.6	2.2	3.0	5.0
12	0.09	0.6	1.2	1.8	2.4	3.9
For drains with "N" = 0.015						
Corrugated Plastic Pipe						
4	0.53	3.3	6.8	9.8	13.3	21.9
5	0.40	2.5	5.1	7.3	9.9	16.6
6	0.32	2.0	4.0	5.8	7.9	13.2
8	0.21	1.3	2.7	3.9	5.3	8.8
10	0.16	1.0	2.0	2.9	4.0	6.6
12	0.13	0.8	1.6	2.3	3.1	5.1

(a) "N" is the roughness coefficient and must be obtained from pipe manufacturer.

Source: SCS, 1973

c. Pipe Diameter

The diameter of the drain pipe is a function of design discharge, hydraulic gradient, and the roughness coefficient. With this information, the appropriate drain diameter can be determined based on the Manning velocity equation (5-14) or from nomographs prepared using the Manning formula. Figure 5-26 is a nomograph for estimating pipe diameter for pipe with an N value of 0.015. A pipe diameter one size larger than that determined to be necessary is generally recommended.

5.2.3.3 Filters and Envelopes

The primary function of a filter is to prevent soil particles from entering and clogging the drain. Filters should always be used where soils have a high percentage of fines.

The function of an envelope is to improve water flow and reduce flow velocity into the drains by providing a material that is more permeable than the surrounding soil. Envelopes may also be used to provide suitable bedding for a drain and to stabilize the soil material on which the drain is being placed. Envelopes are required for most applications.

Although filters and envelopes have distinctly different functions, well graded sands and gravels can be used to meet the requirements of both a filter and an envelope.

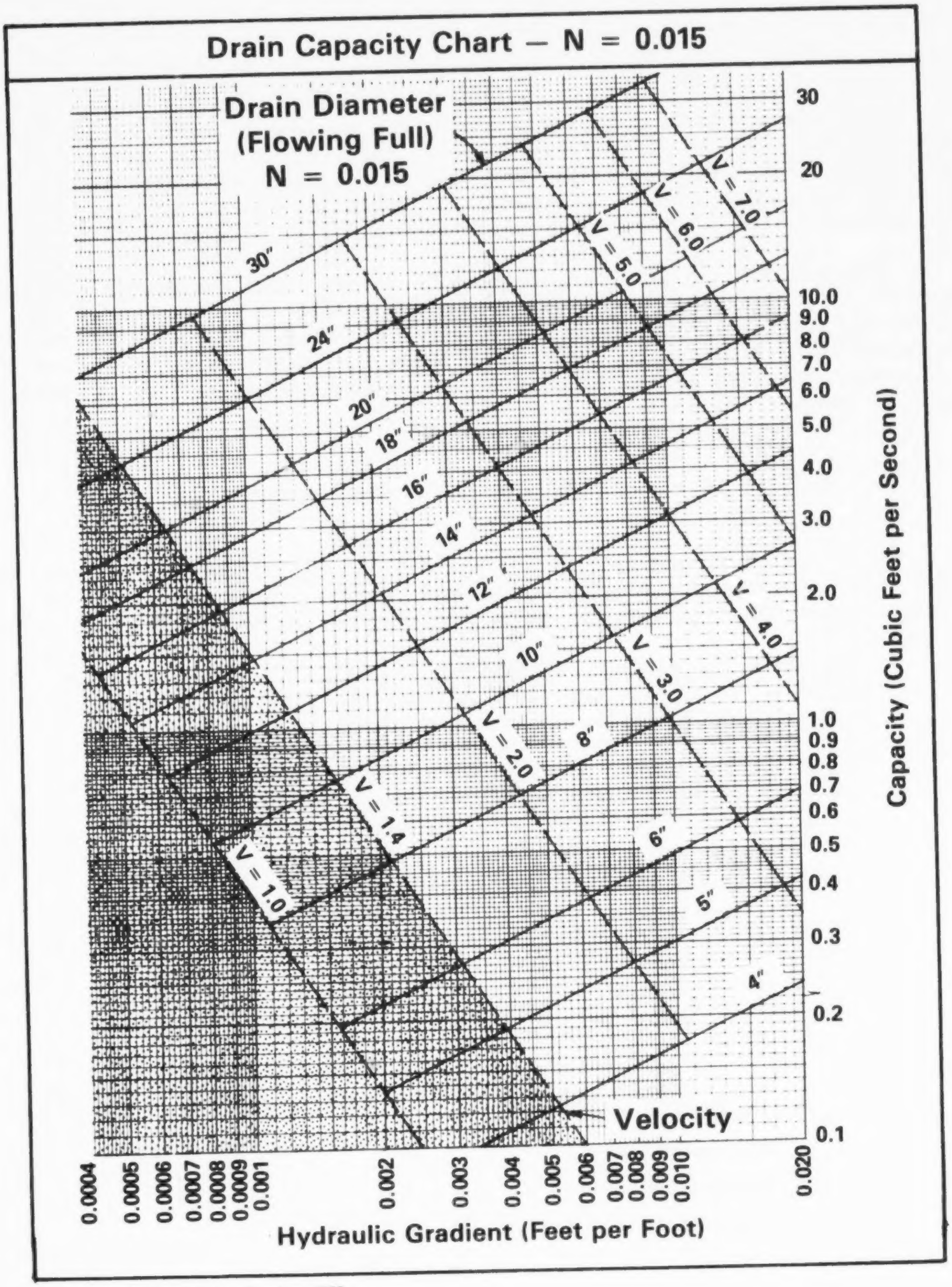
Geotextiles are also widely used as filters. They are generally made of polypropylene, polyethylene, polyester, or polyvinyl chloride. Filter fabric should be selected based on its compatibility with the leachate.

The general procedure for designing a gravel filter is to: (1) make a mechanical analysis of both the soil and the proposed envelope material; (2) compare the two particle distribution curves; and (3) decide by some set of criteria whether the envelope is satisfactory. SCS (1973) and others developed various criteria which set size limits for a filter material based on the size of the base material. A filter is considered satisfactory if it allows some of the fine soil particles to pass through so as not to plug the filter but retains larger particles which would deposit in the drain.

For synthetic materials the suitability of a filter can be determind from the ratio of particle size distribution to the pore size of the fabric.

The first requirement of sand and gravel envelopes is that the envelope have a hydraulic conductivity higher than that of the base material. SCS (1973) generally recommends that all of the envelope material should pass the 1.5-inch sieve, 90 percent should pass the 0.75-inch sieve, and not more than 10 percent should pass the No. 60 sieve (0.01-inch). This minimum limitation is the same for filter materials, however, the gradation of the envelope is not important since it is not designed to act as a filter (SCS, 1973).

FIGURE 5-26.
CAPACITY CHART FOR N=0.015



Source: Soil Conservation Service, 1973

A minimum thickness of four inches is recommended for a gravel envelope (Bureau of Reclamation, 1978).

5.2.3.4 Design of Manholes

Manholes are used in subsurface drainage systems to serve as junction boxes between drains; silt and sand traps; observation wells; and access points for pipe location, inspection, and maintenance. Manholes should be located at junction points, changes in alignment or grade, and other designated points. There are no set criteria for manhole spacing.

A manhole should extend from 12 to 24 inches above the ground surface for ease of location. The base of the manhole should be a minimum of 18 inches below the lowest pipe to provide a trap for sediments. Manholes are typically designed to have a drop in elevation between the inlet and outlet pipes to compensate for head losses in the manhole (Bureau of Reclamation, 1978). A typical manhole design is shown in Figure 5-27.

5.2.3.5 Design of the Sump and Pumping System

Contaminated groundwater is collected by gravity flow in a drainage sump from which it is pumped to treatment (Figure 5-28). The major steps in designing the sump and pumping system include (Bureau of Reclamation, 1978):

- Determine the maximum inflow (Q_p) to the sump: Maximum inflow is based on total discharge (Q). An extra 20 percent allowance is usually made for flows in excess of design discharge.
- Determine the amount of storage required: The cycling operation of the pump determines the amount of storage required. Maximum storage occurs when the inflow rate is one half of the discharge of the pump. Therefore, storage volume (S_v) is equal to one half the cycling time (6 for a 12 minute cycle pump) times Q_p.
- Determine the pumping rate: Pumping rate Q_m (ft^3/min) is determined from the following expression:

$$Q_m = (S_v + Q_p t_p)/t_p \qquad (5\text{-}16)$$

where:

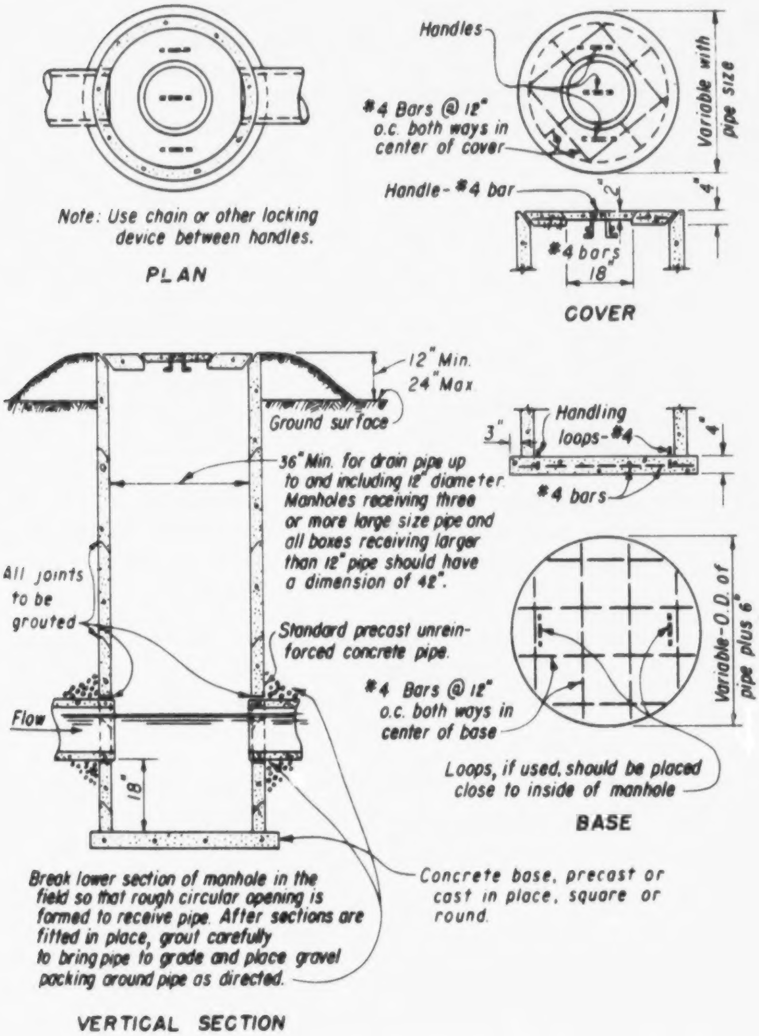
Q_m = pumping rate (ft^3/min)

S_v = storage volume (ft^3)

Qp = maximum inflow (ft^3/min)

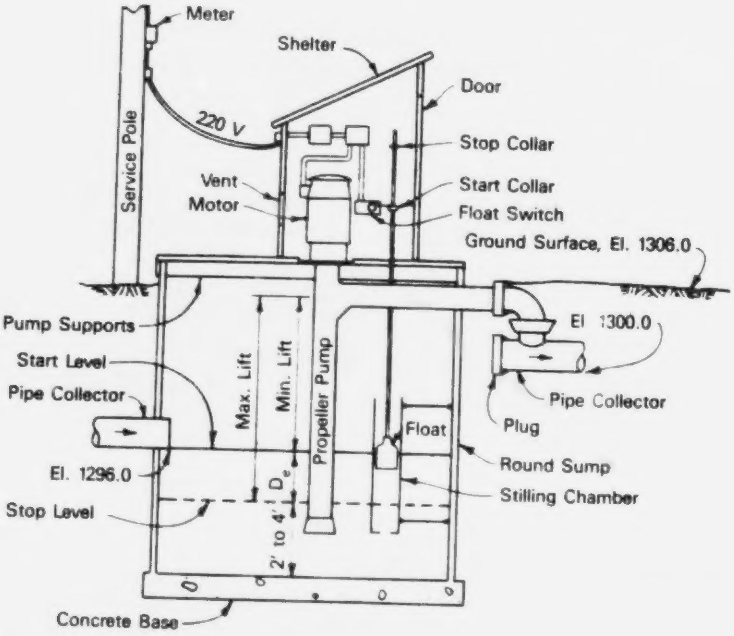
t_p = running time of the pump (min).

FIGURE 5-27.
TYPICAL MANHOLE DESIGN FOR A CLOSED DRAIN


Note: Use chain or other locking device between handles.
PLAN
Handles
#4 Bars @ 12" o.c. both ways in center of cover
Variable with pipe size
Handle- #4 bar
2"
4"
#4 bars
18"
COVER
12" Min. 24" Max
Ground surface
36" Min. for drain pipe up to and including 12" diameter. Manholes receiving three or more large size pipe and all boxes receiving larger than 12" pipe should have a dimension of 42".
All joints to be grouted
Standard precast unreinforced concrete pipe.
Flow
18"
Break lower section of manhole in the field so that rough circular opening is formed to receive pipe. After sections are fitted in place, grout carefully to bring pipe to grade and place gravel packing around pipe as directed.
Concrete base, precast or cast in place, square or round.
VERTICAL SECTION
3"
Handling loops-#4
4"
#4 bars
Variable-O.D. of pipe plus 6"
#4 Bars @ 12" o.c. both ways in center of base
Loops, if used, should be placed close to inside of manhole
BASE

Source: Bureau of Reclamation, 1978

FIGURE 5-28.
TYPICAL DESIGN OF AN AUTOMATIC DRAINAGE PUMPING PLANT



Source: Bureau of Reclamation, 1978

- Determine the start, stop, and discharge levels: In general, the maximum water level for starting the pump should be at about the top of the pipe drain discharging to the sump. The minimum elevation should be about 2 to 4 feet above the base of the sump.

- Determine the size of the sump: The volume required for storage plus the criteria that the minimum water level should be 2 to 4 feet above the bottom of the sump determines the size of the sump.

- Select the pump: As described in Section 5.1, selecting a pump for a particular application requires that the total head capacity be determined. Manufacturers' performance curves can then be used to determine pump efficiency and necessary horsepower. Centrifugal or diaphragm pumps are generally used.

5.2.4 Construction

The major activities associated with construction of subsurface drains are trench excavation, trench stabilization, and installation of the drains and filter and envelope materials.

5.2.4.1 Trench Excavation and Associated Activities

a. Trench Excavation

Trench excavation is one of the most critical elements in determining the cost-effectiveness of drains. The need for extensive rock fragmentation may result in exclusion of drains as a cost-effective remedial action.

Trench excavation is usually accomplished using either trenching machines or backhoes. Cranes, clamshells, and draglines are also used for deep excavation.

Trenchers or ditchers are designed to provide continuous excavation in soil and well-fragmented or weathered rock. They consist of a series of buckets mounted on a wheel (bucket-wheel type) or a chain sprocket and ladder (bucket-ladder type). In continuous trenching, the wheel or ladder is lowered as the revolving buckets excavate the trench to the appropriate depth. The trench assembly may be mounted on wheels or on semi-crawler or full-crawler frames. The trencher moves forward simultaneously as the trench is excavated, resulting in a trench of neat lines and grades. The bucket wheel types are generally used to dig shallow trenches for agricultural drainage. The maximum depth for a large wheel trencher is about 8.5 feet (Church, 1981). Bucket-ladder type trenchers can excavate trenches up to 27 feet deep and about 6 feet wide, although 4 feet is the maximum economical width (Church, 1981). Different sizes of bucket-wheel-type trenchers are available for various depths and widths. Buckets may be changed to fit the type of soil being excavated.

The factors that influence the rate of trenching include: (1) soil moisture; (2) soil characteristics such as hardness, stickiness, stones; and (3) depth and width of trench.

Generally, continuous trenching in suitable materials is much faster than trenching via backhoe. Hourly production rates for wheel and ladder trenches operating at 100 percent efficiency is given in Table 5-9. Actual efficiencies may range from 20 to 90 percent depending upon the above mentioned factors (Church, 1981).

Trenchers can be equipped with back-end modifications to provide shoring, install a geotextile envelope, lay tile or flexible piping, blind the piping, and backfill with gravel or excavated soil.

TABLE 5-9

APPROXIMATE HOURLY PRODUCTION IN CUBIC YARDS FOR LADDER AND WHEEL TRENCHERS OPERATING AT 100 PERCENT EFFICIENCY

	Engine hoursepower of trencher			
Rock-earth formation	50	100	150	200
Alluvium, sand-gravels, lightly cemented	210	420	630	840
Weathered rock-earth:	180	360	540	720
• Maximum weathering	120	240	360	480
• Minimum weathering	60	120	180	240

Source: Church, 1981

backhoes can excavate earth and fragmented rock up to one-half of the bucket diameter to depths of up to 70 to 90 feet.

The crane and clamshell can be used for deeper excavation or when access excludes the use of the backhoe. Use of draglines is generally limited to removal of loose rock and earth. Operation and production rates for backhoes, cranes, and clamshells are described in Section 7.1.

Excavation of a trench through material containing numerous large boulders or hard rock layers results in considerable construction delays and substantially increases the cost of construction. Typically, these materials must be fractured to facilitate their removal.

The most commonly used method for fragmenting rock in hazardous waste site work involves the use of rotary or percussion drills; backhoe-mounted pneumatically driven impact tools (Hobgoblin); and tractor-mounted mechanical rippers. The Hobgoblin has a low production rate of about six cubic yards per hour (Richardson Engineering Services, 1980). Mechanical rippers have considerably higher production rates than the other methods, but they are limited to depths of 6 feet or less and are not suitable for highly consolidated rock. The depth limitation can be overcome to some extent if the ripper can enter the trench to rip lower lifts, but this becomes uneconomical since the trench width clearance increases the volume of material to be excavated. Blasting, though commonly used in the construction industry for rock fragmentation, is not recommended for hazardous waste site work.

b. Grade Control

Proper grade control in a subsurface drain ensures against ponding of water and provides for a nonsilting velocity in the drainage pipe. Proper grade control can be accomplished using either automatic laser or visual grade-control systems. Laser systems are adaptable to a wide range of earth-moving equipment including trenchers and backhoes.

In visual grade control, grade stakes of equal length are driven to the design subgrade at selected points along the trench line. A line drawn through the top of the grade stakes would be parallel to the design grade of the trench. Targets are driven next to the grade stakes and are adjusted to a fixed distance above the elevation of the grade stakes. The selection of this distance depends on the depth of the trench and the line of sight between the machine operator and a reference sighting rod on the machine. When the trenching machine is cutting on grade, the target will align with the reference sighting rod.

Accuracy of plus or minus 0.1 foot is easily obtainable with the target method, although it depends upon the machine operator's skill and alertness. If the depth of the trench is also checked with respect to the design grade at each target point, the target grade control method can attain an accuracy within plus or minus 0.02 foot of the design grade (Taylor and Willardson, 1971).

c. Dewatering

Proper installation of drains (i.e., maintenance of grade, placement and alignment of pipes) generally requires dewatering to achieve a dry environment. Three basic options are available for dewatering: open pumping, predrainage using wellpoints or well systems, and groundwater cutoff. These techniques may be used separately or in combination. Open pumping involves construction of a sump hole or pit at the lowest point of the excavation so that water can flow towards and collect in the pit. A centrifugal submersible pump or a diaphragm pump can then be used to pump the accumulated water from the sump holes. Any contaminated water is subsequently treated. Open pumping is applicable only to shallow trench excavations with stable soils of low hydraulic conductivity where groundwater seepage into the excavation is minimal (Powers, 1981). It is often used together with predrainage where wells or wellpoints have reduced seepage to a manageable volume.

Wellpoints and deep wells can be used to lower the water table near a trench excavation. Wellpoints are one of the most widely used and most versatile dewatering technologies. The use of wellpoints and deep wells and their associated costs are discussed in Section 5.1.

Groundwater cut-off barriers such as steel sheet piling, concrete, or a bentonite slurry may also be used together with wells and wellpoints to reduce the size of the predrainage system required. These methods are described in Section 5.3.

d. Wall Stabilization Methods

Trench excavations generally require the use of wall stabilization methods to prevent cave-ins during installation of drain pipes. With shallow trenches in stable soils, the need for shoring can be eliminated by cutting the trench with sloped walls so that a stable angle is attained (usually a 1.5 [horizontal] to 1 [vertical] slope).

Shoring, which involves supporting the trench wall with wood or steel structures, is the most commonly used method of wall stabilization. Shoring methods for supporting shallow trenches involve the use of slipshields (constructed on-site by welding I-beams between two parallel pieces of sheet steel) and adjustable aluminum bracing.

For trenches which are deeper than about 10 feet, steel sheet piling or steel H-piles with horizontal wooden beams between them can be driven and braced to support the trench walls.

5.2.4.2 Drain Installation

Once trench excavation is completed, the components of the subsurface drain can be installed. This process includes laying the pipes, filter, and envelope material; backfilling; and installation of auxiliary components.

a. Laying the Pipes

All subsurface drains must be laid on a stable bed with the desired grade. Trenches that have inadvertently been overexcavated should be refilled with dry soil and brought to grade with envelope material. Well-graded gravel is then laid in an even layer several inches thick to provide bedding for the pipes. Flexible piping must be installed in a way that ensures firm support for its entire circumference. This may be accomplished by shaping a semicircular groove in the trench bottom so that the gravel fill forms a cradle for the pipe.

Pipe installation begins at the lowest trench elevation and proceeds upgrade. When bell and spigot type pipes are used, the bell end should always be upgrade. During installation, the water level in the trench should not exceed 50 percent of the pipe diameter above the invert of the pipe. Water may be removed by allowing it to flow through previously installed tubing (Bureau of Reclamation, 1978). Alternatively, the trench floor could deliberately be overexcavated and backfilled with a larger size stabilizing gravel. The envelope material can then be laid on top of the stabilized floor.

To take advantage of the characteristics of flexible tubing, equipment capable of automatic pipe installation should be used. Trenching machines can be modified to include a hopper for bedding/envelope material with chutes to deliver the material; a rack for a roll of tubing and/or filter fabric which is designed and located to minimize stretching; and a conveyor for automatic backfill (SCS, 1973).

Rigid pipes cannot be installed automatically and long lengths of pipe are either hand carried or lowered by crane into the trench.

When extending drainage systems under roadways, structures, root zones, or areas not requiring drainage, unperforated pipe should be specified.

b. Placement of Envelope and Filters

Gravel envelopes are installed around the pipe drain to increase flow into the drain and reduce the build up of sediments in the drain line. They may be placed by hand, backhoe, or by a hopper cart or truck. In continuous trencher drain installation machines, gravel filling may be ongoing along with other operations.

Filter fabrics are sometimes installed around the gravel envelope to prevent fines from clogging the envelope and drain pipe. When constructing a drain using a fabric filter wrapping, the fabric is installed first, followed by the bedding, the pipe, and the envelope in that order. The fabric filter is then wrapped around the top of the envelope prior to backfilling with soil. A schematic of an installed pipe drain with filter fabric is shown in Figure 5-17. Fabric filters can be installed manually or by machine.

c. Backfilling

After the gravel envelope has been installed, the trench must be backfilled to the original grade. Prior to backfilling, the drain should be inspected for proper elevation below ground surface; proper grade and alignment; broken pipe; and thickness of the gravel envelope. The inspector should ensure that pipe drains and manholes are free of deposits of mud, sand and gravel, or other foreign matter, and are in good working condition. Unstable soils may preclude all but spot checks before backfilling.

Almost any type of excavation equipment can be used to backfill trenches including backhoes, bulldozers, scrapers, and combination backhoe-front-end loader. During backfilling, care should be taken to ensure that the drain is not disturbed either vertically or horizontally. About 1 foot of fill should be carefully placed over the envelope before starting the general backfill operation (Bureau of Reclamation, 1978).

Geotextile fabric may be used on the top of the envelope to prevent siltation of the envelope from the backfill materials. In order to prevent settling of the backfill after construction, periodic compaction of soil lifts is also required. This may be accomplished using air tamping or a vibrating or sheepsfoot compactor.

5.2.5 Performance Monitoring

After installation of the subsurface drain is complete, the drain should be tested for obstructions. For a small drainage system, this can be done visually by shining a high powered flashlight through a drain from one manhole and observing the beam in another. TV camera inspections may be used for large diameter drains. Mechanical methods can be used both to remove obstructions or to test for obstructions. Flexible polyurethane foam plugs are available which expand to wipe or scrape the pipe when water or air pressure is applied. They are also available with a rope through the center so they can be pulled through the drain pipe (Knapp, Inc., 1982).

Manholes and silt traps should be checked frequently for the first year or two of operation for sediment build up. Less frequent inspection is required as the system ages.

Piezometers may be installed in the various parts of the drainage system to identify operational problems with the filter, envelope, pipe, or other components of the system. Piezometers can measure the loss of head through a medium, and, thus, can identify obstructions to flow, such as a clogged envelope or filter.

Monitoring wells can also be installed downgradient of the drainage system. Detection of contaminants would indicate a malfunction or failure of the system.

Malfunction of subsurface drains can be attributed to chemical clogging, clogging due to biological slimes, or a variety of physical mechanisms. Problems caused by the above conditions are usually apparent at the surface above the drain. Inspection of the area will reveal soft or ponded surface conditions, areas of subsidence, and areas of accelerated vegetative growth.

Chemical clogging of pipes and envelope materials can occur by a number of mechanisms. Calcium carbonate precipitates and iron and manganese deposits can build up around collector pipes or can cause cementation of the envelope material. Factors contributing to calcium carbonate precipitation include bicarbonate alkalinity, calcium hardness, pH, and changes in pressure or partial pressure of carbon dioxide. Formation of iron and manganese deposits depend upon redox potential, iron and manganese concentrations, total sulfides, presence of iron reducing bacteria, pH, and the presence of materials which form soluble (e.g, Cl^-, CN^-) or insoluble (phenols, humic acids) iron complexes. Presence of viscous wastes can also clog drain envelope materials.

Physical mechanisms which result in drain malfunction include formation of sinkholes or blowouts due to pipe breakage, root penetration, high pressure within the drains, and hydraulic removal of fines (scouring). Excessive scouring can result from improper design of envelope material or from excessive velocity. A certain amount of scour is to be expected. If manholes are not cleaned out periodically, sediments can build up to the point of clogging the drains.

Clogged drain pipes can be corrected using high pressure water jetting equipment, mechanical scrapers or brushes, or flexible foam plugs, as described previously. Flexible foam plugs are coated with plastic material in a spiral or criss-cross design which imparts added scouring power to the plug. The plastic can be impregnated with silicon carbonate or steel bristles to remove scale or deposits (Knapp, Inc., 1982). In some cases, chemicals may be needed to remove difficult deposits.

Where there is a structural problem, such as drain breakage or improper drain spacing causing a sinkhole, the drain must be dug up and the condition corrected. Malfunctioning perforated pipe drains located near root systems should be dug up and replaced with non-perforated pipe.

5.2.6 Technology Selection/Evaluation

Evaluation of the suitability of subsurface drains as a remedial technology is generally made by comparing the cost-effectiveness of this alternative with pumping. Relative to pumping, subsurface drains can be difficult and costly to install particularly where extensive hard rock excavation and dewatering is required. They are also time consuming to install and may not be an appropriate alternative where immediate remediation is required. Safety of field workers is also more of a concern with subsurface drains because of the need for extensive trench excavation.

However, there are several advantages of drains relative to pumping. They are generally more cost-effective than pumping in areas with low hydraulic conductivity particularly where pumping would be required for an extended period of time. They are easier to operate since water is collected by gravity flow. They are also more reliable from the standpoint that there are no electrical components which can fail. However, when drains fail due to clogging, breaks in the pipes, or sinkhole formation, they can be costly and time consuming to rehabilitate.

5.2.7 Costs

Costs for installation and operation of subsurface drains can be divided into four categories: installation costs, materials costs, engineering supervision, and operation and maintenance.

Installation costs depend primarily on the depth of excavation, stability of soils, extent of rock fragmentation required, and groundwater flow rates. The principal materials costs include pipes, gravel, manholes, and pumps and other accessories for the drainage sump. Materials and installation unit costs have been combined and are summarized in Tables 5-10 through 5-12.

Engineering and supervision involves such activities as staking the drain line, checking for grade control and alignment; and checking pipe specification and pipe quality, etc. For the installation of subsurface drains in conventional agricultural and water conservation applications, engineering and supervision costs are usually about 5 to 10 percent of the total (Schwab et al., 1981). However, these costs can be expected to be substantially higher for hazardous waste site applications and will vary considerably depending upon the geologic and hydrogeologic conditions.

Capital costs associated with installation of subsurface drains are typically much higher than those associated with pumping systems. This is particularly true where substantial rock excavation is required and deep drains requiring extensive shoring are needed. These factors may result in exclusion of drains as a cost-effective remedial action. However, operation and maintenance costs associated with drains are generally lower than with pumping, provided the system is properly designed and maintained. Lower operation and maintenance costs become significant particularly where plume removal or containment is needed over a long period of time.

Total capital costs for drainage systems, as with other remedial technologies, can vary widely depending upon site conditions. Two scenarios are briefly described below to illustrate how widely capital costs may range.

At one particular hazardous waste site (Site A) a 261 foot long interceptor drain was installed to a depth of 12 to 17 feet. The leachate discharged into a 4 foot-wide, 20 foot deep sump which pumped the leachate at a rate of 18 to 20 gpm to a treatment system (USEPA, 1984) (USEPA, 1985).

Construction of the drainage system involved excavation to a 4 to 6 foot wide trench which was supported with steel sheet piling during construction. The trench was lined with filter fabric, 6 inches of gravel and a 12 inch perforated concrete asbestos drain pipe. Additional filter fabric supported by screening was then wrapped around the pipe prior to backfilling. The total cost of the drainage system was $269,721 or $65 to 86/square foot, (USEPA, 1984) (USEPA, 1985).

Table 5-13 shows how the total capital costs were distributed.

A second case history involved installation of a shallow (3 foot deep) interceptor trench at the A.W. Mauthe site in Appleton, Wisconsin (USEPA 1984).

The drainage system was approximately 750 feet long and consisted of 4 inch PVC pH drainage pipe which was laid in a gravel filled trench. Four foot diameter concrete sumps were installed at two collection points and the sumps were connected by about 25 feet of PVC pipe so that water collected in

TABLE 5-10
1985 UNIT COSTS FOR TRENCH EXCAVATION AND ASSOCIATED ACTIVITIES

Item	Assumptions	Unit Cost	Source
Trench Excavation			
Trencher, ladder type	gas, 5 ft deep, 8-in wide	\$485/day	(1)
	diesel, 8 ft deep, 16-in wide	\$650/day	(1)
Backhoe, hydraulic	4 ft wide trench, damp sandy loam		
	1 yd^3 capacity, 12 ft deep; 90 LF/day	\$2.20/yd^3	(2)
	1.25 yd^3 capacity, 14 ft deep; 90 LF/day	\$2.11/yd^3	(2)
	2.5 yd^3 capacity; 18 ft deep; 115 LF/day	\$1.69/yd^3	(2)
	3.5 yd^3 capacity; 20 ft deep; 136 LF/day	\$1.40/yd^3	(2)
Dragline	1.75 yd^3 capacity, 35 yd^3/hr	\$3.00/yd^3	(1)
	1.5 yd^3 capacity, 65 yd^3/hr	\$1.93/yd^3	(1)
Clamshell	0.5 yd^3 capacity, 20 yd^3/hr	\$4.83/yd^3	(1)
	1.0 yd^3 capacity, 35 yd^3/hr	\$3.23/yd^3	(1)
Rock Fragmentation			
Jackhammer	---	\$39/hr	(2)
Track-mounted air drill	---	\$74/hr	(2)

(continued)

LF = linear feet

TABLE 5-10 (continued)

Item	Assumptions	Unit Cost	Source
Wall Stabilization (continued)			
Sheet piling	includes pull and salvage:		
	15 ft excavation, 22 $\mathrm{lb/ft}^2$	$\$7.15/\mathrm{ft}^2$	(1)
	25 ft excavation, 38 $\mathrm{lb/ft}^2$	$\$8.30/\mathrm{ft}^2$	(1)
	40 ft excavation, 38 $\mathrm{lb/ft}^2$	$\$7.70/\mathrm{ft}^2$	(1)
Wooden shoring	includes wales, braces, and spacers; pull and salvage:		
	14 ft excavation	$\$5.80/\mathrm{ft}^2$	(1)
	20 ft excavation	$\$6.75/\mathrm{ft}^2$	(1)
H-piles	H-piles with 3-in wood sheeting horizontal between piles; includes removal of wales and braces:		
	15 to 22 feet	$\$19$ to $22/\mathrm{ft}^3$	(1)
	23 to 35 feet	$\$21$ to $24/\mathrm{ft}^3$	(1)
	36 to 45 feet	$\$24$ to $27/\mathrm{ft}^3$	(1)
	53 to 57 feet	$\$29$ to $31/\mathrm{ft}^3$	(3)
Dewatering			
Sump hole	includes excavation and gravel:		
	with 12-in corrugated pipe	$\$19.70/\mathrm{ft}^3$	(1)
	with 15-in corrugated pipe	$\$25/\mathrm{ft}^3$	(1)
	with 18-in corrugated pipe	$\$28/\mathrm{ft}^3$	(1)
	with 24-in corrugated pipe	$\$38/\mathrm{ft}^3$	(1)

(continued)

TABLE 5-10 (continued)

Item	Assumptions	Unit Cost	Source
Dewatering (continued)			
Opening pumping	Pumping 8 hrs, attended 8 hrs; includes 20 feet of suction hose and 100 feet of discharge hose:		
	2-in diaphragm pump	$355/day	(1)
	4-in diaphragm pump	$390/day	(1)
	3-in centrifugal pump	$356/day	(1)
	6-in centrifugal pump	$430/day	(1)
Submersible centrifugal sump pump	Bronze, without installation:		
	1/4 hp, 22 gpm, 10 ft head	$200 each	(1)
	1/2 hp, 68 gpm, 10 ft head	$325 each	(1)
	1/2 hp, 94 gpm, 10 ft head	$430 each	(1)
	Cast iron, without installation:		
	1/4 hp, 23 gpm, 10 ft head	$ 94 each	(1)
	1/3 hp, 35 gpm, 10 ft head	$105 each	(1)
	1/2 hp, 68 gpm, 10 ft head	$220 each	(1)
Diaphragm pump	Cast iron starter and level control, without installation; 2-in discharge:		
	10 gpm, 20 ft head	$305 each	(1)
	60 gpm, 20 ft head	$400 each	(1)
	120 gpm, 20 ft head	$750 each	(1)
	160 gpm, 20 ft head	$1,175 each	(1)

(continued)

TABLE 5-10 (continued)

Item	Assumptions	Unit Cost	Source
Wellpoint dewatering	See Section 5.1 for costs	—	
Groundwater cutoffs	See Sheet Piling, above, for costs	—	
Grade Control			
Automatic laser control		$140/day	(1)

(1) Godfrey, 1984a.
(2) Godfrey, 1984b.
(3) McMahon, 1985a.

TABLE 5-11
1985 UNIT COSTS FOR PIPE INSTALLATION

Item	Assumptions	Unit Cost	Source
Drain Pipe			
PVC perforated underdrain	10 foot length, S.D.R. 35:		
	4 inch	\$2,162/ft	(2)
	6 inch	\$3.64/ft	(2)
	8 inch	\$4.56/ft	(2)
	10 inch	\$6.80/ft	(2)
	12 inch	\$8.40/ft	(2)
Corrugated steel or aluminum, perforated, asphalt coated	6 inch, 18 ga	\$4.63/ft	(1)
	8 inch, 16 ga	\$6.20/ft	(1)
	10 inch, 16 ga	\$8.00/ft	(1)
Porous wall concrete underdrain, extra strength	6 inch	\$4.14/ft	(1)
	8 inch	\$5.80/ft	(1)
	10 inch	\$8.75/ft	(1)
Vitrified clay, extra heavy duty strength, premium joints	4 inch	\$4.46/ft	(2)
	5 inch	\$5.35/ft	(2)
	6 inch	\$6.35/ft	(2)
	8 inch	\$8.50/ft	(2)
Filter and Envelope			
Filter fabric	Polypropylene, laid in trench	\$1.14-1.49/yd^2	(2)
Gravel envelope	Crushed bank run, screened 0.75-0.50-in; in trench	\$9.20-10.55/yd^3	(1)
Backfill			
Dozer backfill, no compaction	Up to 300 foot haul, 900 yd^3/day	\$1.11/yd^3	(1)
Dozer backfill, air tamped	Up to 300 foot haul, 235 yd^3/day	\$5.45/yd^3	(1)

(continued)

TABLE 5-11. (continued)

Item	Assumptions	Unit Cost	Source
Backfill (continued)			
Compacted backfill, vibrating roller	6 to 12 inch lifts, 700 yd^3/day	\$1.54/yd^3	(1)
Compacted backfill, sheepsfoot roller	6 to 12 inch lifts, 650 yd^3/day	\$1.67/yd^3	(1)

Sources: (1) Godfrey, 1984a; (2) Godfrey, 1984b.

TABLE 5-12
1985 INSTALLED COSTS FOR MANHOLES

Item		Cost
Concrete slab, cast in place, 8" thick	6-ft deep	$890
	8-ft deep	$1,275
	12-ft deep	$1,915
	16-ft deep	$2,555
	20-ft deep	$3,195
Pre-cast concrete riser pipe, 4 ft inside diameter	6-ft deep	$570
	8-ft deep	$775
	12-ft deep	$1,175
	16-ft deep	$1,575
	20-ft deep	$1,975
6 ft inside diameter	6-ft deep	$1,250
	8-ft deep	$1,675
	12-ft deep	$2,535
	16-ft deep	$3,395
	20-ft deep	$4,255
Slab tops, precast, 8-in thick	4-ft diameter	$175
	5-ft diameter	$195
	6-ft diameter	$270
Frames and covers, watertight	24-in diameter	$345
	32-in diameter	$430

Source: Godfrey, 1984a

TABLE 5-13. CAPITAL COSTS ($1985) FOR INTERCEPTOR DRAIN INSTALLATION - SITE A

Materials		
• 550 feet of 12 inch perforated asbestos cement drainage pipe, (only about 261 feet were actually used)		$3,806
• 147 feet of 2 inch carbon steel pipe for carrying leachate to treatment system		1,549
• 2 submersible pumps and accessories		1,041
• 2,700 ft^2 vinyl-coated wire screen		702
• 338 yd^2 filter fabric		277
• Other materials costs not given		4,342
	Subtotal	11,717
Labor/Equipment		
Labor, equipment rental including excavation equipment and sheet piling and gravel fill		$257,342
Company in-house labor		651
	TOTAL	269,721

Source: Adapted from USEPA, 1984

one sump could be pumped to the other. The total capital cost for the drainage system was about $15,400 (updated to 1985 costs using ENR Construction Cost Indeces for 1982 and 1985). However, this cost estimate also includes cost for a 300 foot long surface water diversion drainpipe. Therefore, the unit cost for the subsurface drain was less than \$6.86/ft^2 (USEPA, 1984).

These two case histories show an order of magnitude difference in the unit cost for subsurface drainage systems. Probably the most significant factor contributing to these cost differences is the depth of the drain. In the case of Site A the trench was excavated to a depth of 12 to 17 feet and shoring was required to support the trench wall. In the case of the Mauthe site the subsurface drainage system was only 3 feet deep and trench excavation was greatly simplified.

5.3 Subsurface Barriers

The term subsurface barriers refers to a variety of methods whereby low permeability cut-off walls or diversions are installed below ground to contain, capture, or redirect groundwater flow in the vicinity of a site. The most commonly used subsurface barriers are slurry walls, particularly soil-bentonite slurry walls. Less common are cement-bentonite or concrete (diaphragm) slurry walls, grouted barriers, and sheet piling cut-offs. Grouting may also be used to create horizontal barriers for sealing the bottom of contaminating sites. These types of subsurface barriers are discussed in the following sections.

5.3.1 Slurry Walls

Slurry walls are the most common subsurface barriers because they are a relatively inexpensive means of vastly reducing groundwater flow in unconsolidated earth materials. The term slurry wall can be applied to a variety of barriers all having one thing in common: they are all constructed in a vertical trench that is excavated under a slurry. This slurry, usually a mixture of bentonite and water, acts essentially like a drilling fluid. It hydraulically shores the trench to prevent collapse, and, at the same time, forms a filter cake on the trench walls to prevent high fluid losses into the surrounding ground. Slurry wall types are differentiated by the materials used to backill the slurry trench. Most commonly, an engineered soil mixture is blended with the bentonite slurry and placed in the trench to form a soil-bentonite (SB) slurry wall. In some cases, the trench is excavated under a slurry of portland cement, bentonite, and water, and this mixture is left in the trench to harden into a cement-bentonite (CB) slurry wall. In the rare case where great strength is required of a subsurface barrier, pre-cast or cast-in- place concrete panels are constructed in the trench to form a diaphragm wall. These types of slurry walls, including hybrids of the three, are discussed below.

5.3.1.1 Soil-Bentonite Slurry Walls

Soil-bentonite slurry walls are backfilled with soil materials mixed with a bentonite and water slurry. Of the three major types of slurry walls, soil-bentonite walls offer the lowest installation costs, the widest range of chemical compatibilities, and the lowest permeabilities. At the same time, soil-bentonite walls have the highest compressibility (least strength), require a large work area, and, because the slurry and backfill can flow, are applicable only to sites that can be graded to nearly level (Spooner et al., 1984a). The following sections present details on applicability, design, construction, monitoring, evaluation, and costs for soil-bentonite slurry walls.

a. Applications and Limitations

One of the first steps in considering a soil-bentonite slurry wall for a given site is to review the configuration options available and determine which best meets the goals of the remedial action. In the vertical perspective, slurry walls may be "keyed-in" or hanging, as shown in Figure 5-29. Keyed-in slurry walls are constructed in a trench that has been excavated into a low permeability confining layer such as a clay deposit or competent bedrock. This layer will form the bottom of the contained site, and a good key-in is essential to adequate containment. Hanging slurry walls, however, are not tied to a confining layer but extend down several feet into the water table to act as a barrier to floating contaminants (such as oils and fuels) or migrating gases. The use of hanging slurry walls in waste site remediation is relatively rare, and most installations are of the keyed-in variety.

In the horizontal perspective, slurry walls can be placed (relative to the direction of groundwater flow) upgradient, downgradient, or completely surrounding the waste site. The various horizontal placement options are shown in Figure 5-29.

Circumferential installations are by far the most common and offer several advantages. This placement vastly reduces the amount of uncontaminated groundwater entering the site from upgradient, thus reducing the volume of leachate generated. Also, provided there are no compatibility problems between the site wastes and the wall materials (discussed later), the amount of leachate leaving the downgradient side of the site will be greatly reduced. Moreover, when this configuration is used in conjunction with an infiltration barrier and a leachate collection system (or other means of reducing the hydraulic head on the interior of the wall), the hydraulic gradient can be maintained in an inward direction, thus preventing leachate escape.

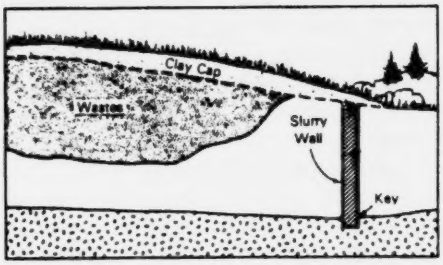
Upgradient placement of a slurry wall refers to one installed on the groundwater source side of the wastes. Although no documented cases of upgradient installatons were found, this placement could be used to divert clean groundwater around a site in high gradient situations. This method will not halt the generation of leachate but could slow its generation by stagnating groundwater behind the wall.

Downgradient placement refers to installation of a slurry wall on the side of a waste site toward which groundwater is flowing. Although not common, this placement can be employed as a hanging wall to contain and capture floating contaminants and methane. Because there is direct leachate/wall contact with this configuration, extensive compatibility testing is essential.

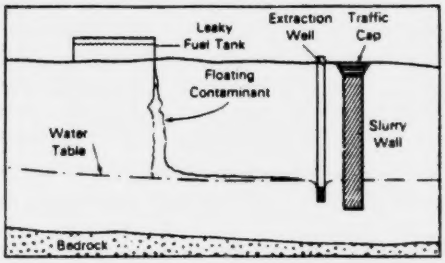
Another major concern in the application of soil-bentonite walls to site remediation is the compatibility of the backfill mixture with site contaminants. Evidence indicates that soil-bentonite backfills are not able to withstand attack by strong acids and bases, strong salt solutions, and some organic chemicals (D'Appolonia, 1980b). The issue of chemical incompatibility is discussed further under "Design Considerations."

FIGURE 5-29.
EXAMPLES OF SLURRY WALL PLACEMENT OPTIONS

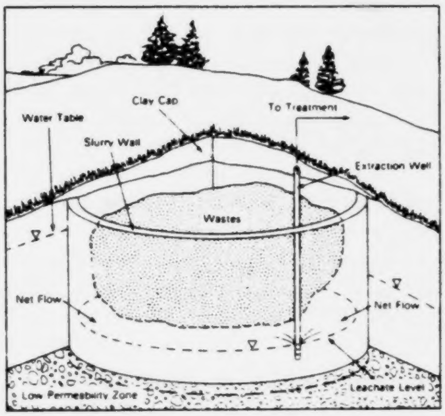
Keyed-in Slurry Wall



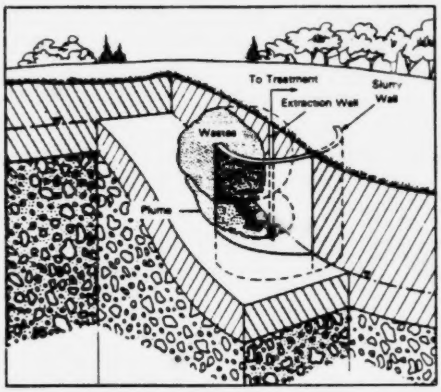
Hanging Slurry Wall



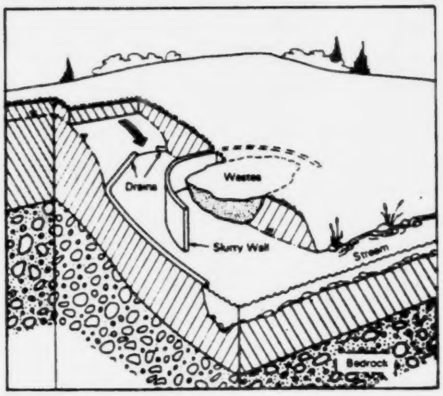
Cut-away Cross-section of Circumferential Wall Placement



Cut-away Cross-section of Downgradient Placement



Cut-away Cross-section of Upgradient Placement with Drain



Source: Spooner et al., 1984a

A number of factors can limit the application of soil-bentonite to a particular site. Most limitations can be overcome by increased engineering, but the associated cost increase may make some other alternative, such as groundwater pumping, a more suitable remedial measure.

One factor that can limit the use of a soil-bentonite wall is the site topography. Because both the excavation slurry and the backfill will flow under stress, the trench line must be within a few degrees of level. In most cases, it is possible to grade the trench line level prior to construction, but this is an added expense.

Cement-bentonite slurry walls are an alternative for steeply sloping sites and are discussed in Section 5.3.1.2.

If a keyed-in slurry wall is considered, the depth to, and nature of, the confining layer becomes a concern. The layer must be of sufficiently low permeability to significantly retard downward migration at the design head levels. It must also have sufficient thickness to allow for excavation of an adequate key-in (2 to 3 feet). The depth to the confining layer will also determine the type of excavation equipment used and the completed wall costs. Most slurry wall contractors have available modified hydraulic backhoes capable of excavation to depths of seventy feet or more. Below this level, more expensive specality equipment such as clamshell grabs are required, and costs increase dramatically.

Another limiting factor is the amount of work area required for soil-bentonite backfill mixing. Ideally, there will be sufficient work area beside the trench to mix and place the backfill. If such room is available, a central backfill mixing area can be established, and the backfill hauled to the active trench portion.

Another limiting factor in the use of soil-bentonite slurry walls for pollution migration control is the lack of long-term performance data. Soil-bentonite walls have been used for decades for groundwater control in conjunction with large dam projects and there is ample evidence of their success in this application. However, the ability of these walls to withstand long-term permeation by many contaminants is in question. Most contaminant/ backfill compatibility questions have been answered by laboratory permeation tests and not by long-term field studies. The issue of containment/backfill compatibility is discussed further under "Design Considerations."

b. Design Considerations

A host of factors must be considered in the design of a slurry wall. The design must be based on a detailed, design-phase investigation characterizing subsurface conditions and materials as well as waste disposition and nature. The issue of waste/wall compatibility should be addressed early in the design by permeability testing of the proposed backfill mixture with actual site leachate or groundwater. The design-phase investigation results can then be used to decide on the optimum configuration and to select any ancillary measures needed to enhance the performance of the wall. These considerations are discussed further below. These and other design considerations are covered in great detail in Slurry Trench Construction for Pollution Migration Control (Spooner et al., 1984a).

For most slurry walls installed to control comtaminated groundwater, the most important design consideration will be the permeability of the completed wall. The soil-bentonite wall's permeability is dependent on the backfill mixture. The lowest permeability is obtainable from backfills having 20 to 40 percent fine soil material (passing a number 200 sieve) and preferably plastic fines (D'Appolonia, 1980b). Although plastic fines are essential to achieving low permeability, there is evidence to show that long-term wall performance in a contaminated environment will be more certain if the backfill is composed of clays with a low activity index (defined as the plasticity index divided by the percent by weight finer than two microns) (Alcar, Oliveri, and Field, 1984).

If a backfill mixture contains clays with a high activity index, the initial low permeability of the wall could more easily be increased by physico-chemical reactions brought about by contaminants. This sort of problem can be avoided by proper backfill selection and testing.

A number of chemical compounds can have a detrimental effect on soil-bentonite slurry walls. Table 5-14 shows how some chemicals can effect backfill permeability. More recent information indicates that organic fluids can cause dessication and cracking in soil-bentonite backfill mixtures, and result in permeability increases of several orders of magnitude. However, these same data indicate that these organics, at or near their solubility limits in aqueous solution, caused no appreciable increases in permeability (Evans, Fang, and Kugelman, 1985). Nonetheless, landfill and lagoon leachates are often complex mixtures of chemicals and no pollution control slurry wall should be installed without thorough compatibility testing.

After it is determined that a backfill mixture compatible with site wastes can be designed, an assessment can be made on wall configuration. In most pollution migration control applications, the wall will be keyed into a low permeability confining layer beneath the site, and completely surround the site. Only in special applications are hanging walls and partial walls used.

The design of a slurry wall for source control at a site must always consider how the wall fits into the overall remedial response. Rarely, if ever, are slurry walls (or other subsurface barriers) the only action taken in site remediation. At a minimum, surface infiltration barriers (caps) are installed to prevent filling of the site interior and overtopping of the wall. In some installations, extraction wells or drains are used to maintain lower groundwater levels inside the wall than outside. This prevents the site from filling and possibly floating the cover material off, and also keeps groundwater flowing towards the interior, thus permeating the wall with groundwater rather than leachate. Although such an installation would be more expensive and would require a leachate treatment system, long-term performance of the wall would be greatly enhanced.

TABLE 5-14
SOIL BENTONITE PERMEABILTIY INCREASES DUE TO LEACHING WITH VARIOUS POLLUTANTS

Pollutant	Backfill$^+$
Ca^{++} or Mg^{++} @ 1,000 ppm	N
Ca^{++} or Mg^{++} @ 10,000 ppm	M
NH_4NO_3 @ 10,000 ppm	M
Acid (pH>1)	N
Strong acid (pH<1)	M/H*
Base (pH<11)	N/M
Strong base (pH>11)	M/H*
HCl (1%)	N
H_2SO_4 (1%)	N
HCl (5%)	M/H*
NaOH (1%)	M
CaOH (1%)	M
NaOH (5%)	M/H*
Benzene	N
Phenol solution	N
Sea water	N/M
Brine (SG=1.2)	M
Acid mine drainage ($FeSO_4$, pH ~ 3)	N
Lignin (in Ca^{++} solution)	N
Organic residues from pesticide manufacture	N
Alcohol	M/H

N - No significant effect; permeability increase by about a factor of 2 or less at steady state.
M - Moderate effect; permeability increase by factor of 2 to 5 at steady state.
H - Permeability increase by factor of 5 to 10.
* - Significant dissolution likely.
+ - Silty or clayey sand, 30 to 40% fines.

Sources: D'Appolonia, 1980a; D'Appolonia and Ryan, 1979

c. Construction Considerations

Construction of a soil-bentonite slurry wall is relatively straightforward. The equipment used is dependent on the depth and length of the wall. For walls up to 80 feet deep, a backhoe or modified backhoe is used for excavation. Deeper installations require the use of a mechanical or hydraulic clamshell or, in rare cases, a dragline. Small volume wall installations may allow the use of batch slurry and backfill mixing systems, while large jobs would require flash slurry mixers and a large backfill mixing area. Figure 5-30 illustrates a typical slurry wall construction site.

Regardless of the equipment used, the slurry is introduced just after the trench is opened and before the water table is reached. The primary function of the slurry is to act as hydraulic shoring to prevent trench collapse. There is also evidence to indicate that the filter cake formed on the trench walls by the slurry, contributes to the low permeability of the completed wall.

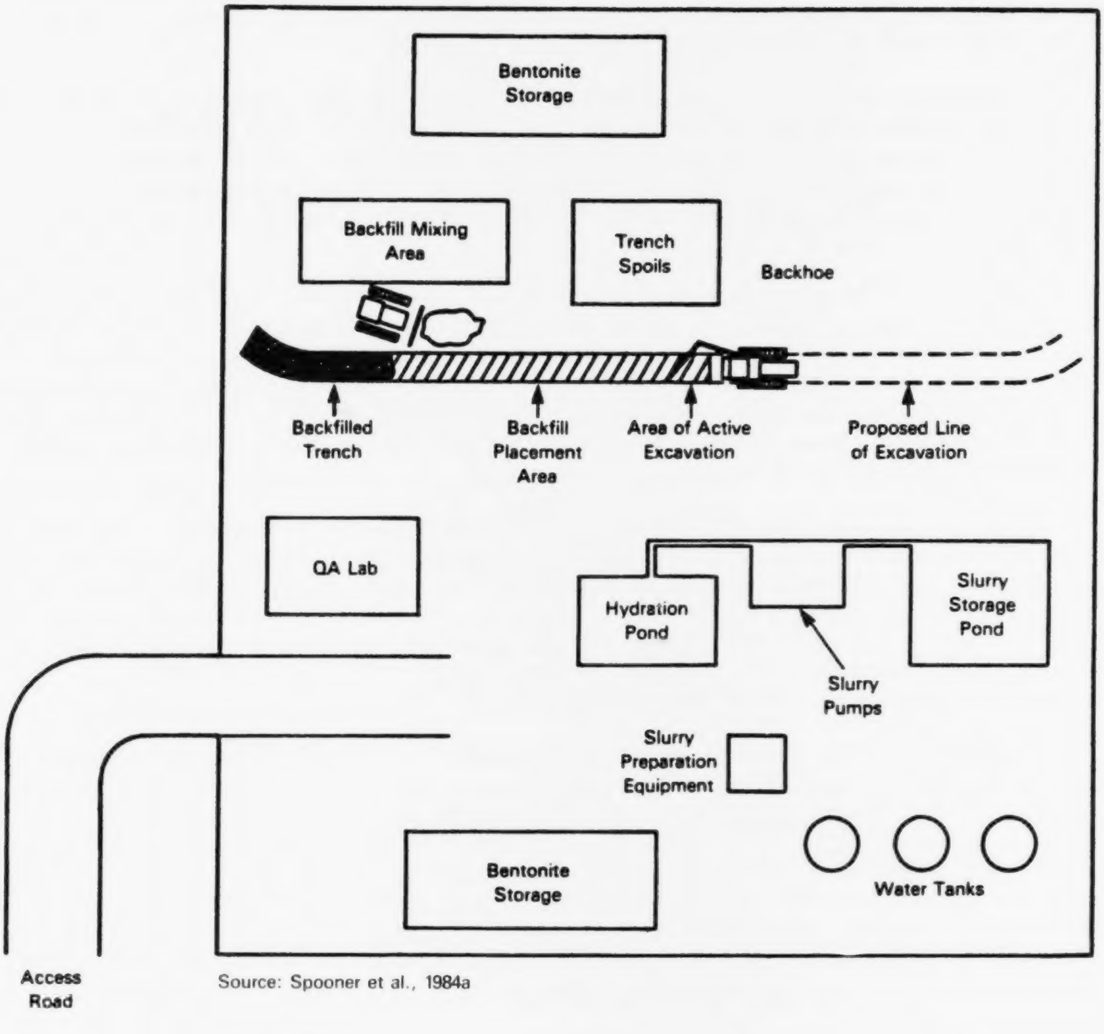
After a sufficient length of wall is excavated to the design depth, backfilling can begin. This is usually begun by using a clamshell to lower mixed backfill to the trench bottom until the sloped backfill extends to the surface. Thereafter, the backfill can be pushed into the trench with a bulldozer or poured from trucks using a trough, and allowed to flow (not fall) down the sloped backfill. This procedure, with backfill mixing alongside the trench, is shown in Figure 5-31.

Proper quality control during wall installation is essential. The most important factors are checks of trench continuity and backfill mixing and placement. For backhoe-excavated trenches, the continuity of the trench is relatively easy to verify. Inspection of the excavated material indicates when and where the confining layer is encountered, and observation of the motion of the backhoe arm confirms lateral continuity. With clamshell excavators, confirmation of lateral continuity may be more complicated.

Backfill mixing and placement are carefully controlled during construction. The soil which makes up the majority of the backfill is placed in the mixing area (either a central area or alongside the trench) and slurry is added. The mixture is bladed and tracked by a bulldozer until it is of relatively uniform consistency and of the proper density. The backfill must be fluid enough to flow freely into the trench, but not so fluid that it: (1) fails to easily displace the slurry; (2) forms a slope so gentle that it extends into the active excavation area; or (3) a great length of trench must be kept open.

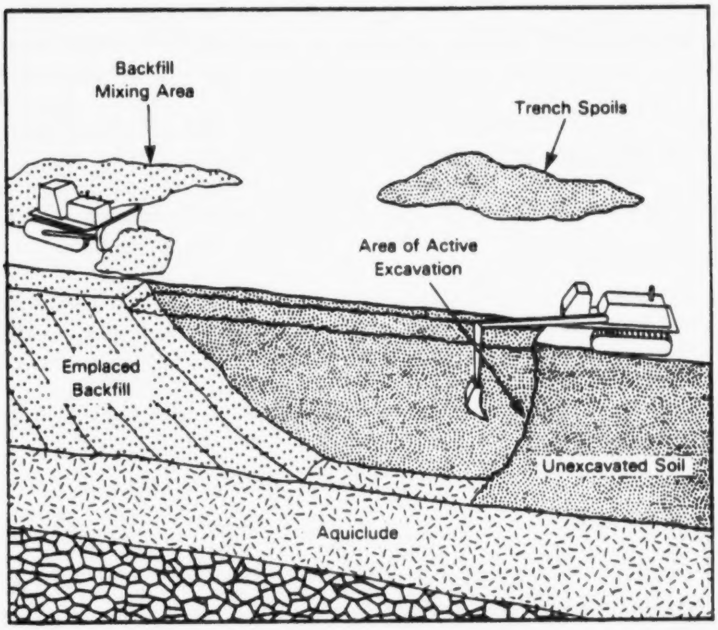
For circumferential installation, a portion of the backfill is re-excavated when the circle is complete. The wall is then allowed to consolidate for up to several weeks. Dessication and consolidation cracks often form in the top few feet during this time. These are often excavated and a compacted earth cap placed along the wall to prevent further dessication or cracking. Where vehicular traffic must cross a wall, traffic caps of

FIGURE 5-30.
TYPICAL SLURRY WALL CONSTRUCTION SITE



Source: Spooner et al., 1984a

FIGURE 5-31.
CROSS-SECTION OF SLURRY TRENCH, SHOWING EXCAVATION AND BACKFILLING OPERATIONS



Source: Spooner et al., 1984a

aggregate and/or geotextiles are often constructed. Dessication caps and traffic caps are tied into the site surface cap.

d. Operation, Maintenance, and Monitoring

As passive measures, slurry walls require no operation and little maintenance. Maintenance of the dessication cap atop the wall is the only requirement that is specific to the wall itself. Maintenance of ancillary measures such as caps and leachate collection systems is important to the wall as part of the entire remedy. Monitoring of slurry walls usually involves monitoring groundwater levels inside and outside the wall to ensure that design head levels are not exceeded. Groundwater quality monitoring can be used to determine the effectiveness of the entire remedial effort.

e. Technology Selection/Evaluaton

Soil-bentonite slurry walls are a relatively inexpensive and effective means of controlling groundwater flow. They have been in use for decades controlling seepage through, under, and around large dams. In uncontaminated environments, they have been shown to have long-term effectiveness and require little or no maintenance. Although they are installed by specialty contractors, they are relatively easy to construct and are effective in controlling groundwater immediately, provided head differentials across the wall are within design tolerances. In contaminated environments, however, their effectiveness over the long-term is very dependent on the types of contaminants and their concentrations. Consequently, design of such installations should always consider methods and measures of minimizing direct contact of high strength leachates with the wall. The integrity of any slurry wall placed directly through wastes or kept in constant contact with high strength leachates must be questioned, and where questioned, verified by monitoring.

The major safety concerns for slurry wall installations arise from the excavation of contaminated materials. These can cause disposal problems, increase air emissions from the site, and greatly slow the pace of construction by requiring increased levels of worker protection (i.e., supplied air). This is another reason that excavations through deposited wastes should be avoided whenever possible.

f. Costs

Costs for slurry walls and other subsurface barriers are usually expressed in costs per unit area of wall (dollars per square foot). Thus, total costs are determined by the depth and the length. Width is determined by the excavation equipment being used. Table 5-15 shows average costs for soil-bentonite and cement-bentonite slurry walls, and illustrates the effects of both the depth and ease of excavation on costs. Operation and maintenance costs are negligible. These costs have been updated using Engineering News Record cost indices for 1979 and 1984.

5.3.1.2 Cement-Bentonite Slurry Walls

Cement-bentonite slurry walls share many characteristics with soil-bentonite slurry walls, but are also different in some respects. The principal difference between the two is the backfill, and this produces differences in applications, compatibilities, and costs. To avoid duplication of previous sections, this discussion will highlight the factors that distinguish cement-bentonite walls from soil-bentonite walls.

Cement-bentonite walls are generally excavated using a slurry of Portland cement, bentonite, and water. This slurry is left in the trench and allowed to set up to form the completed barrier. For extremely deep installations, a

TABLE 5-15
RELATION OF SLURRY CUT-OFF WALL COSTS PER SQUARE FOOT AS A FUNCTION OF MEDIUM AND DEPTH

	Slurry Trench Prices in 1984 Dollars Soil-Bentonite Backfill ($\$/\text{ft}^2$)			Unreinforced Slurry Wall Prices in 1984 Dollars Cement-Bentonite Backfill ($\$/\text{ft}^2$)		
Medium	Depth ≤ 30 Feet	Depth 30-75 Feet	Depth 75-120 Feet	Depth ≤ 60 Feet	Depth 60-150 Feet	Depth > 150 Feet
Soft to Medium Soil $N \leq 40$	3-6	6-11	11-14	21-28	28-42	42-104
Hard Soil N 40 - 200	6-10	7-14	14-28	35-42	42-55	55-132
Occasional Boulders	6-11	7-11	11-35	28-42	42-55	55-118
Soft to Medium Rock, Sandstone, Shale, $N \geq 200$	9-17	14-28	28-69	69-83	83-118	118-243
Boulder Strata	21-35	21-35	69-111	42-55	85-132	132-292
Hard Rock Granite, Gneiss, Schist*	---	---	---	132-194	194-243	243-326

Notes:

N is standard penetration value in number of blows of the hammer per foot of penetration (ASTM D1586-67)

*Normal Penetration Only:
For standard reinforcement add $\$11/\text{ft}^2$.
For construction in urban environment add 25% to 50% of price.
Does not include cost of working in a contaminated environment.

Updated from: Ressi di Cervia, 1980

normal bentonite slurry is used for excavation, then replaced by cement-bentonite.

a. Applications and Limitations

Cement-bentonite walls offer the same configuration options as soil-bentonite walls. They are more versatile than soil-bentonite walls in two ways. First, because the slurry sets up into a semi-rigid solid, this type of wall can accommodate variations in topography by allowing one section to set while continuing the next section at a higher or lower elevation. Second, because the excavation slurry is commonly the backfill too, this type of wall is better suited to restricted areas where there is no room to mix soil-bentonite backfill. Also, cement-bentonite is stronger than soil-bentonite and so is used where the wall must have less elasticity, such as adjacent to buildings or roads.

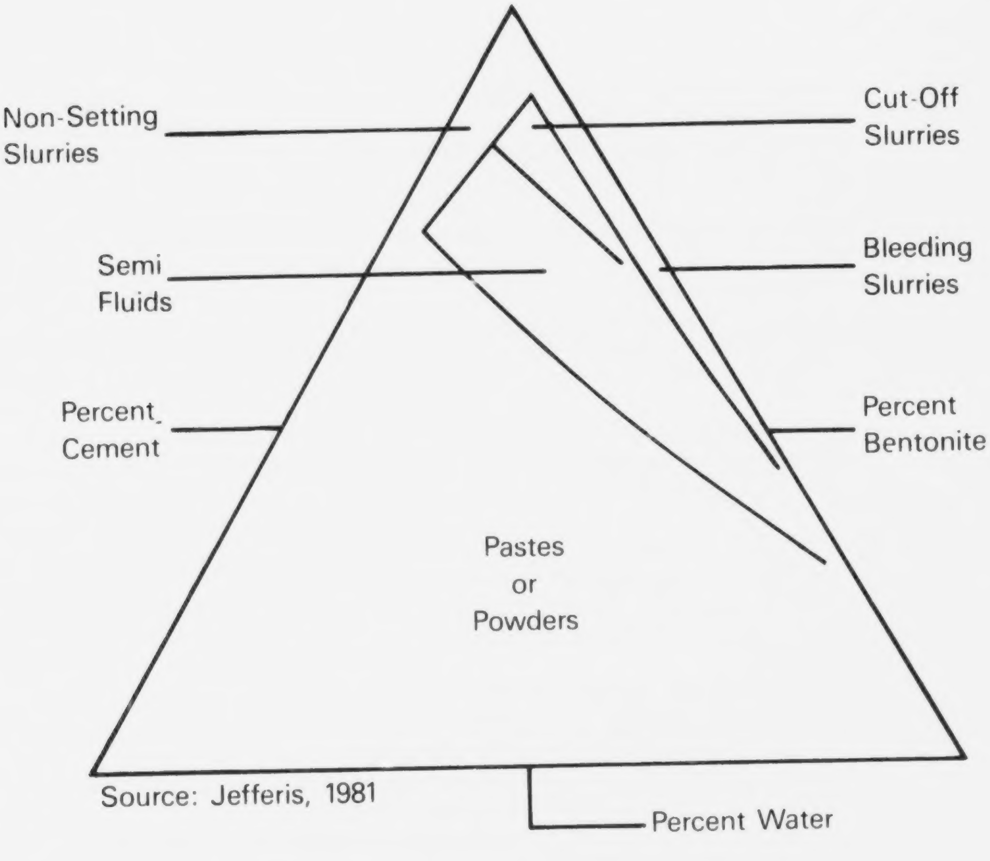
Cement-bentonite slurry walls are limited in their use by their higher costs, somewhat higher permeability, and their narrower range of chemical compatibilities. As Table 5-16 illustrates, cement-bentonite walls average over 30 percent higher in cost than soil-bentonite walls. The permeability of a cement-bentonite wall is normally around 1×10^{-6} cm/sec, while a well designed soil-bentonite wall is capable of achieving 1×10^{-8} cm/sec (Spooner et al., 1984a). Cement-bentonite backfills are also more susceptible to chemical attack than most soil-bentonite mixtures. Cement-bentonite is susceptible to attack by sulfates, strong acids and bases (pH ≤ 4 and ≥ 7), and other highly ionic substances. A more complete discussion of compatibilities and compatibility testing is contained in *Compatibility of Grouts with Hazardous Wastes* (Spooner et al., 1984b).

b. Design and Construction Considerations

The design and construction of a cement-bentonite slurry wall is very similar to that of a soil-bentonite wall. Table 5-16 shows typical compositions of cement-bentonite slurries. Accelerators, retardants, and various other additives may be used but are not common practice. Figure 5-32 illustrates the composition of cement-bentonite slurries.

Construction of a cement-bentonite wall is nearly identical as for a soil-bentonite wall. Although backfill mixing is eliminated, along with the large area requirement, greater care must be taken in slurry mixing because the mixture is more sensitive to small changes in composition. Cement-bentonite walls are also usually finished with dessication caps to prevent harmful cracking.

FIGURE 5-32.
COMPOSITION OF CEMENT-BENTONITE SLURRIES



Source: Jefferis, 1981

TABLE 5-16
TYPICAL COMPOSITIONS OF CEMENT BENTONITE SLURRIES

Constituent	Percentage in Slurry
Bentonite	4-7
Water	68-88
Cement	
without replacements	8-25
when blast furnace slag added, minimums	1-3
when fly ash added, minimums	2-7
Blast furnace slag, maximums, if used	7-22
Fly ash, maximums, if used	6-18

Adapted from: Jefferis, 1981.

c. Operation, Maintenance, and Monitoring

As with soil-bentonite walls, there is no operation required. Maintenance and monitoring are usually directed toward ancillary measures, and not toward the slurry wall itself. Monitoring water levels and groundwater quality is important, however, in evaluating the success of the wall as part of the remedial effort.

d. Technology Selection/Evaluation

Like soil-bentonite slurry walls, cement-bentonite walls can be an effective, relatively inexpensive means of controlling groundwater flow. Most of the site constraints that affect the selection of a soil-bentonite wall also apply to cement-bentonite walls. Because cement-bentonite walls are more expensive, they are generally used where: (1) there is no room to mix and place soil-bentonite backfill, (2) increased strength is required, or (3) extreme topography make it impractical to grade a site level.

As with any barrier installation, thorough compatibility testing is a must. Cement-bentonite mixtures are somewhat more susceptible to chemical attack than most soil-bentonites, and should not be placed directly through wastes or left unprotected from attack by high-strength leachates.

e. Costs

Average costs for cement-bentonite slurry walls and the effect of depth and digging ease on costs are shown in Table 5-15. They are more expensive than soil-bentonite walls due mainly to the cost of Portland cement. It should be noted that these costs do not reflect work in a hazardous environment.

5.3.1.3 Diaphragm Walls

Diaphragm walls are barriers composed of reinforced concerete panels (diaphragms), which are emplaced by slurry trenching techniques. They may be cast-in-place or pre-cast, and are capable of supporting great loads. This degree of strength is seldom if ever called for at a hazardous waste site and their use is extremely rare. Because diaphragm walls are constructed in slurry-filled trenches, it is possible to include them in cement-bentonite or soil-bentonite walls for short sections, such as road or rail crossings, that require their greater strength. Provided the joints between the cast panels are made correctly, diaphragm walls can be expected to have permeabilities comparable to cement-bentonite walls. The same compatibility concerns that apply to cement-bentonite, apply to diaphragm walls.

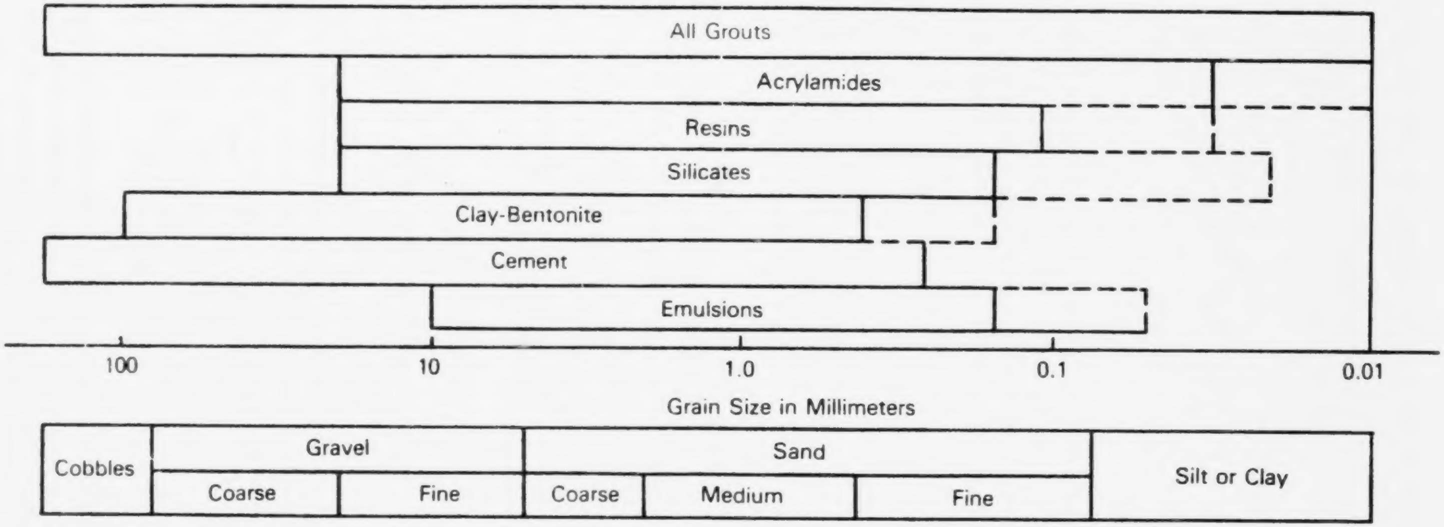
Because diaphragm walls are a specialty item, and rarely used in pollution migration situations, they are not discussed in detail here. As shown in Table 5-15, diaphragm walls cost approximately $11/square foot more than cement-bentonite walls.

5.3.2 Grouting

Grouting refers to a process whereby one of a variety of fluids is injected into a rock or soil mass where it is set in place to reduce water flow and strengthen the formation. Because of costs, grouted barriers are seldom used for containing groundwater flow in unconsolidated materials around hazardous waste sites. Slurry walls are less costly and have lower permeability than grouted barriers. Consequently, for waste site remediation, grouting is best suited for sealing voids in rock. Even in cases where rock voids are transmitting large water volumes, a grout can be formulated to set before it is washed out of the formation. The various types of grouts available are discussed below followed by discussions of the various ways grouts may be employed. Figure 5-33 illustrates the range of grout applicability based on grain size.

Cement has probably been used longer than any other type of material for grouting applications (Bowen, 1981). Cement grouts utilize hydraulic cement which sets, hardens, and does not disintegrate in water (Kirk-Othmer, 1979). Because of their large particle size, cement grouts are more suitable for rock than for soil applications (Bowen, 1981). However, the addition of clay or

FIGURE 5-33.
APPLICABILITY OF DIFFERENT CLASSES OF GROUTS
BASED ON SOIL GRAIN SIZE



Adapted from: Bowen, 1981

chemical polymers can improve the range of usage. Cement grouts have been used for both soil consolidation and water cut-off applications, but their use is primarily restricted to more open soils. Typically, cement grouts cannot be used in fine-grained soils with cracks less than 0.1 millimeter wide (Bowen, 1981).

Clays have been widely used as grouts, either alone or in formulations because they are inexpensive (Guertin and McTigue, 1982). Only certain types of clay minerals possess the physical and chemical characteristics favorable for use in grouting. These characteristics include the ability to swell in the presence of water and to form a gel structure at low solution concentrations. These properties are possessed most markedly by the montmorillonites. Other types of clay minerals, such as kaolinite and illite, can be used as fillers in grout formulations, such as clay-cement mixtures (Greenwood and Raffle, 1963).

Bentonite grouts (high in calcium montmorillonite) can be used alone as void sealers in coarse sands with a permeability of more than 10^2 ft/day (10^{-1} cm/sec). Bentonite-chemical grouts can be used on medium to fine sands with a permeability between 10^2 ft/day (10^{-1} cm/sec) and 1 ft/day (10^{-3} cm/sec). Both of these grout types can also be utilized to seal small rock fissures (Guertin and McTigue, 1982). Because of their low gel strengths, bentonite grouts are not able to support structures and therefore can only be used as void sealers (Tallard and Caron, 1977b).

Alkali silicates are the largest and most widely used type of chemical grouts. Sodium, potassium, and lithium silicates are available, with sodium silicates being used more frequently. Chemical grouts (i.e., silicates and organic polymers) constitute less than 5 percent by volume of the grouts used in the United States, although they represent almost 50 percent of the grouts used in Europe (Kirk-Othmer, 1979). In addition to their use as a grout, sodium silicates may be used as additives to other grouts, such as Portland cement, to improve strength and durability.

Silicate grouts are used for both soil consolidation and void sealing applications. These grouts are suitable for subsurface applications in soils with a permeability less than 10 ft/day (10^{-2} cm/sec). Silicate grouts are not suitable for open fissures or highly permeable materials because of syneresis (water expulsion) unless they are preceded by cement grouting (Karol, 1982a; Sommerer and Kitchens, 1980). Furthermore, tests conducted by the U.S. Army Waterways Experiment Station found silicate grouts to be ineffective in waterproofing fine-grained soils (Hurley and Thornburn, 1971).

Organic polymer grouts represent only a small fraction of the grouts in use. These grouts consist of organic materials (monomers) that polymerize and crosslink to form an insoluble gel. The organic polymer grouts include:

- Acrylamide grouts
- Phenolic grouts
- Urethane grouts

- Urea-formaldehyde grouts
- Epoxy grouts
- Polyester grouts.

Acrylamide grouts have been in use for about 30 years, and were the first of the organic chemical polymer grouts to be developed. Acrylamide grouts have the largest use among the organic polymer grouts and are the second most widely used chemical grouts (after silicates) (Karol, 1982a). They may be used alone or in combination with other grouts such as silicates, bitumens, clay, or cement (Tallard and Caron, 1977a). Acrylic and polyacrylamide grouts are typically used in ground surface treatment, ground treatment for oil well drilling, and subsurface applications (e.g., waterproof concrete structures). Acrylate grouts are more commonly used for ground surface treatment than for soil injection where acrylamide grouts are more frequently used. Acrylamide applications include structural support and seepage control for mines, soil consolidation for foundations of structures and dams, and water control and soil consolidation for tunnels, wells, and mines (Tallard and Caron, 1977a). Specific applications include grout curtains, loose sand stabilization, artesian flow shut-off, and water seepage control in jointed and fissured rock (Office of the Chief of Engineers, 1973). Based on AM-9 applications, acrylamide grouts may be used in a variety of soil materials such as fine gravel; coarse, medium, or fine sand; coarse silt; and some clays (Herndon and Lenahan, 1976).

Urethane grouts are the second most commonly used type of organic polymer grout (Jacques, 1981). Urethane grouts were developed in Germany for consolidation applications and are now used in Europe, South Africa, Australia, and Japan (Sommerer and Kitchens, 1980). These grouts are used for water and soil applications and can penetrate finely fissured material.

The use of phenolic resin grouts in underground and foundation construction began in the 1960s (Kirk-Othmer, 1979; Tallard and Caron, 1977a). These grouts may be used in fine soils and sands for a variety of water control and ground treatment applications. However, phenolic grouts are not widely used alone but are typically used in conjunction with other grouts (Tallard and Caron, 1977a).

Urea-formaldehyde resins are frequently referred to as aminoplasts. The idea for the use of these resins as grouts came from their use as glue in the oil industry (Tallard and Caron, 1977a). Although urea-formaldehyde grouts have been available since the 1960s, they have found limited usage (Karol, 1982b; Sommerer and Kitchens, 1980). These grouts can set-up only in an acid environment, therefore, they cannot be used in basic formations.

Epoxy grouts and other glue-like grouts have been in use since 1960. These grouts have had limited use in soil grouting primarily because of their high cost (Tallard and Caron, 1977a). Most of the applications reported in the literature involve the use of epoxy resins in mortars and for sealing cracks. Epoxy resins can adhere to and seal submerged concrete, steel, or wood surfaces, and are useful in water applications (Engineering News-Record, 1965). They have been used for grouting cracked concrete for structural repairs and grouting fractured rock to improve its strength (Office of the Chief of Engineers, 1973).

Polyester grouts have been in use since the 1960s. They have been used in a variety of construction applications, principally to treat cracks in buildings and structures (Tallard and Caron, 1977a). These grouts have also been used in mines as well as to stabilize and strengthen porous and fissured rock (Tallard and Caron, 1977a; Office of the Chief of Engineers, 1973). Polyester grouts have been used infrequently to treat sand (Tallard and Caron, 1977a).

The compatibility of these grouts with hazardous wastes and leachates has not been studied in great detail, and only general incompatibilities are known. One recent study indicates the response of several grouts to various organic chemicals is unpredictable and often drastic (Bodocsi, 1985). In any case where grouting is considered a remedial option, thorough compatibility testing must be performed.

The component parts of some grouts, such as acrilimides and urea-formaldehydes, are toxic. Unless the setting reactions are carefully controlled, there is a liklihood that unreacted, toxic compounds will be released into the ground. A thourough characterization of the waste and grout chemistry, as well as the site geochemistry, is required.

5.3.2.1 Rock Grouting

One of the greatest potential uses for grouting in hazardous waste site remediation is for sealing fractures, fissures, solution cavities, or other voids in rock. Nonetheless, rock grouting at waste sites is uncommon and no actual applications were found in the literature.

a. Applications and Limitations

Rock grouting may be applied to a waste site to control the flow of groundwater entering a site. In theory, grouting could also control leachate flow in rock, yet in many cases contaminants interfere with grout setting reactions and/or reduce grout durability. In many cases, the waste/grout interaction and compatibility cannot be predicted and extensive testing is required. These issues are discussed in detail in Compatibility of Grouts with Hazardous Wastes (Spooner et al., 1984b).

b. Design and Construction Considerations

As with other types of barrier construction, the ultimate success of a grouting project depends on thorough site characterization. The ability to seal water bearing voids or zones is dependent on being able to locate them. In many remedial grouting operations, only a small portion of the rock mass will transport water and must be sealed. Consequently, the exploratory investigation must be very thorough. Detailed geologic mapping of the site, aided by remote sensing techniques and extensive rock coring, is required. Even with extensive investigation, the complexity of groundwater flow in fractured and fissured bedrock can make a grouting project impossible to plan completely in advance.

Based on background and exploratory data, the location for a pattern of primary injection holes is chosen and injection at one or more zones is identified. The first few primary holes are then drilled and pressure washed with water and air (Millet and Engelhardt, 1982). This step removes drill cuttings and other debris from the hole to allow better grout penetration. Each hole is then pressure tested, often using a non-setting fluid of the same viscosity as the grout to be used. These tests are used to determine the initial grout mixture and are often conducted using the grout plant and other equipment to be used for the actual grouting (Millet and Engelhardt, 1982 and Karol, 1982a).

Each zone within each primary hole is then injected with the grout mixture until a predetermined amount is pumped (grout take), or a predetermined flow rate at maximum allowable pressure is reached. Maximum allowable pressure is typically around 1 pound per square inch (psi) per foot of overburden (Millet and Engelhardt, 1982). Data from the drilling and injection of the first primary holes is analyzed, and if necessary, the grout mixture or injection pressure modified before completing the remaining primary holes. Following completion of the primary hole grouting, the program is again analyzed, necessary changes made, and a pattern of more closely spaced secondary holes drilled and injected.

The analysis and evaluation of the completed grouting becomes, in essence, another pressure test. Close quality control during drilling and grouting identifies areas that require tertiary hole grouting to complete sealing. Such areas are identified by faster than expected drilling rates and higher than expected grout takes (Millet and Engelhardt, 1982). For a successful grouting program, each hole series (i.e., primary, secondary) will have lower grout takes than the previous one. Many projects will require that proof holes be drilled and injected. A very low grout take on tertiary or proof holes indicates that most voids are grout filled and the grouting program was successful.

c. Operation, Maintenance, and Monitoring

As with other subsurface barriers, operation and maintenance requirements are negligible. Monitoring programs, consisting of hydrologic measurements and water quality assessments, are used to evaluate the effectiveness of the completed barrier.

d. Technology Selection/Evaluation

Rock grouting is very much a specialty operation. It is performed by a limited number of contractors, and each such program is highly site-specific. Because this technique has rarely, if ever, been applied to controlling highly contaminated groundwater, an assessment of performance and reliability is not possible. Each instance where rock grouting is feasible for site remediation must be evaluated on a case-by-case basis.

e. Costs

Each rock grouting job is highly site specific, and valid costs vary widely. Example costs for some common grouts are shown in Table 5-17. These have been updated using the Engineering News-Record cost indices for 1979 and 1985. As shown, individual grout costs can vary widely. Grout costs for a completed job show much less variation. This is because the cheaper, particulate grouts are used to seal large voids, thus using more grout, while the more expensive chemical grouts are commonly used to seal small voids. As an example of costs for rock grouting, assume that a 1,000 foot long barrier, 6 feet thick, and 30 feet deep is to be placed in rock with 20 percent void space. A double row of injection holes, 6 feet on center, will be used (333 holes), and 40 percent sodium silicate grout will be injected. Approximate costs are given in Table 5-18.

5.3.2.2 Grout Curtains

Grout curtains are subsurface barriers created in unconsolidated materials by pressure injection. The various methods of forming a grout curtain are described below under design and construction considerations.

Grout barriers can be many times more costly as slurry walls and are generally incapable of attaining truly low permeabilities in unconsolidated materials. A recent field test study of two chemical grouts revealed signficant problems in forming a continuous grout barrier due to non-coalescence of grout pods in adjacent holes and grout shrinkage. This study concludes that conventional injection grouting is incapable of forming a

TABLE 5-17.
APPROXIMATE COSTS OF COMMON GROUTS

Grout Type	Approximate Cost ($/gallon) of Solution (1985)
Portland cement	1.33
Bentonite	1.76
Silicate - 20%	1.76
- 30%	2.95
- 40%	3.86
Epoxy	42.15
Acrylamide	9.34
Urea formaldehyde	8.00

Adapted from Spooner et al., 1984b.

TABLE 5-18.
APPROXIMATE COSTS ($ 1985) FOR GROUND BARRIER IN ROCK

Unit Operation	Approximate Unit Cost	Approximate Cost
Injection hole drilling	$14.16 per foot 9990 feet	$141,440
Grout Pipe	$8.49 per foot 9990 feet	$84,869
Grout Injection	$5.66 per cubic yard 1333 cubic yards	$7,549
Grout - 40%	$3.86 per gallon sodium silicate 35,991 gallons	$139,039
		Total Cost* $391,097

Source: Updated from USEPA, 1982 using ENR Construction Cost Index

*Does not include site investigation and characterization reliable barrier in medium sands (May et al., 1985). Therefore, they are rarely used when groundwater control in unconsolidated materials is desired.

a. Applications and Limitations

Grout curtains, like other barriers, can be applied to a site in various configurations. Circumferential placement offers the most complete containment but requires that grouting take place in contaminated groundwater downgradient of the source. As discussed under rock grouting, this could easily cause problems with grout set and durability. As with other techniques, this requires extensive compatibility testing during the feasibility study. Another limitation of curtain grouting is the problem of gaps left in the curtain due to nonpenetration of the grout. Only a few small gaps in an otherwise low permeability curtain can increase its overall permeability significantly.

b. Design and Construction Considerations

The design of a grout curtain must be based on a thorough site characterization. Analysis of site characterization data, including boring logs, pump or injection test results, and other data, are used to determine if a site is groutable and which grout is most suitable based on viscosity, compatibility, and ultimate permeability. This is a very involved process and should be conducted by an experienced engineer.

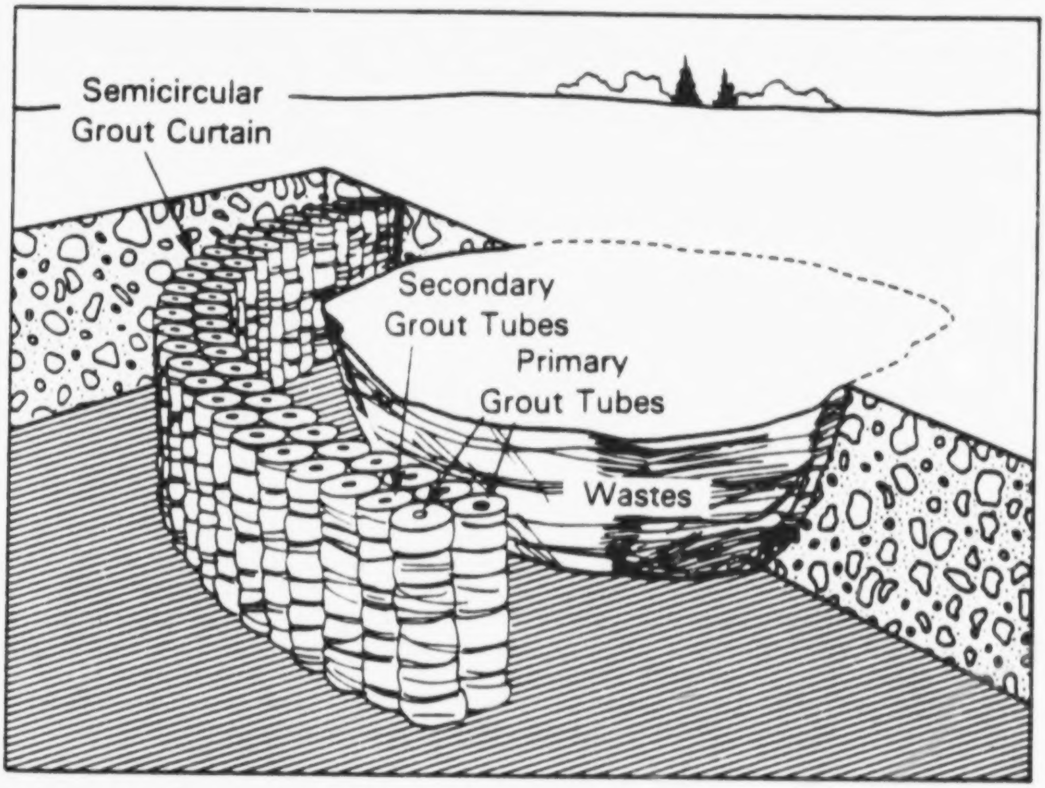
Construction of a grout barrier is accomplished by pressure injecting the grouting material through a pipe into the strata to be waterproofed. The injection points are usually arranged in a triple line of primary and secondary grout holes. A predetermined quantity of grout is pumped into the primary holes. After the grout in the primary holes has had time to gel, the secondary holes are injected. The secondary grout holes are intended to fill in any gaps left by the primary grout injection (Hayward Baker, 1980). The primary holes are typically spaced at 20- to 40-foot intervals (Guertin and McTigue, 1982). Figure 5-34 illustrates a grout curtain.

There are several basic techniques that are utilized to form the grout wall. These include (Hayward Baker, 1980; Guertin and McTigue, 1982):

- Stage-up method
- Stage-down method
- Grout port method
- Vibrating beam method.

In the stage-up method, the borehole is drilled to the full depth of the wall prior to grout injection. The drill is withdrawn one "stage," leaving

FIGURE 5-34
SEMICIRCULAR GROUT CURTAIN AROUND WASTE SITE



Source: Spooner et al., 1984b

several feet of borehole exposed. Grout is then injected into this length of open borehole until the desired volume has been injected. When injection is complete the drill is withdrawn further and the next stage is injected (Hayward Baker, 1980).

Stage-down grouting differs from stage-up grouting in that the injections are made from the top down. Thus, the borehole is drilled through the first zone that is to be grouted, the drill is withdrawn, and the grout injected. Upon completion of the injection, the borehole is redrilled through the grouted layer into the next zone to be grouted and the process is repeated (Guertin and McTigue, 1982).

The grout port method utilizes a slotted injection pipe that has been sealed into the borehole with a brittle Portland cement and clay mortar jacket. Rubber sleeves cover the outside of each slit (or port) permitting grout to flow only out of the pipe. The injection process begins by isolating the grout port in the zone to be injected using a double packer. A brief pulse of high pressure water is injected into the port to rupture the mortar jacket. Grout is pumped between the double packers, passes through the ports in the pipe, under the rubber sleeve, and out through the cracked mortar jacket into the soil (Guertin and McTigue, 1982).

The vibrating beam method is not an injection technique as described above, but instead is a way of placing grout so as to generate a wall. In this method, an I-beam is vibrated into the soil to the desired depth and then raised at a controlled rate. As the beam is raised, grout is pumped through a set of nozzles mounted in the beam's base filling the newly formed cavity. When the cavity is completely filled, the beam is moved less than one beam width along the wall, leaving a suitable overlap to ensure continuity (Harr, Diamond, and Schmednecht, undated). This method is illustrated in Figure 5-35.

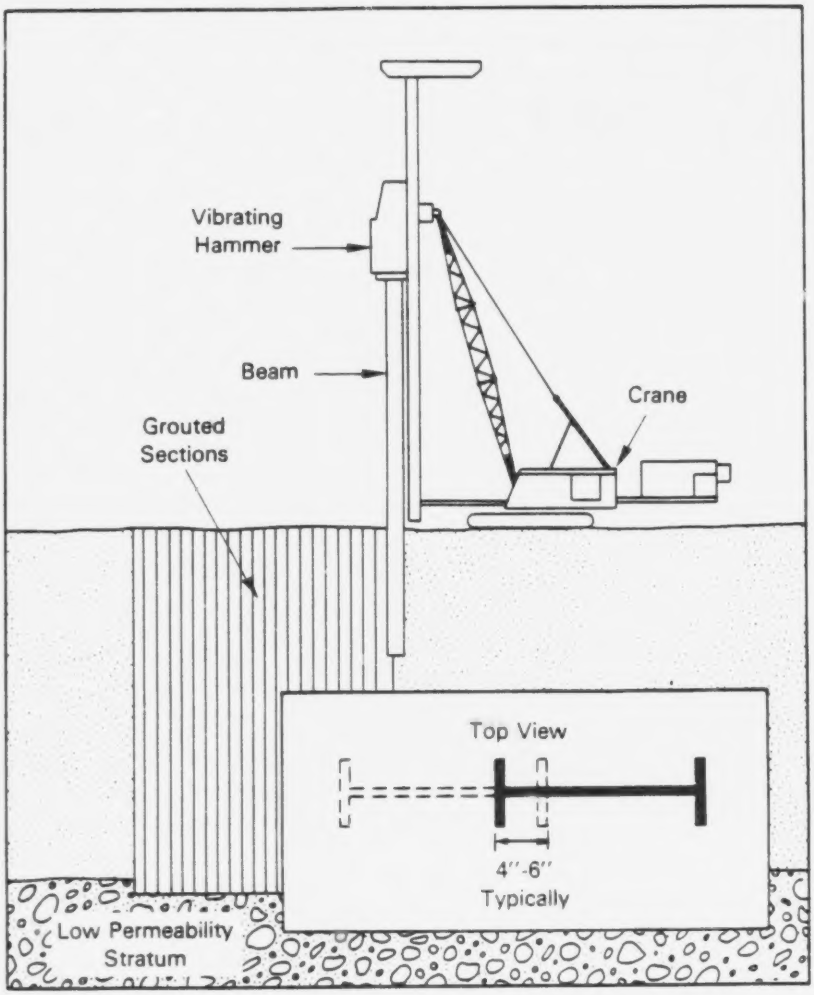
c. Operation, Maintenance, and Monitoring

Grout curtains, while requiring no operation and little or no maintenance may require more monitoring than other barriers. This is because if even a very small gap is left in the barrier, it can enlarge quite rapidly by piping or tunneling if there is a sufficient hydraulic gradient across the wall.

d. Technology Selection/Evaluation

Grout curtains are a specialty technology seldom applied to hazardous waste sites. (This is excepting the vibrating beam wall with bitumen grout, which has seen some application in recent years.) As such, no detailed assessment of the performance or reliability of this technology is possible.

FIGURE 5-35.
VIBRATING BEAM GROUT INJECTION



Source: Soletanche (undated)

e. Costs

As with rock grouting, each curtain grouting job is highly site specific. Each site requires differing degrees of investigation and characterization. The grout injection program must be tailored to its characteristics. The costs given for rock grouting (Section 5.3.2.1) can serve as an example of curtain grouting. Example grout costs are shown in Table 5-17.

5.3.3 Sheet Piling

In addition to slurry wall and grouted cut-offs, sheet piling can be used to form a groundwater barrier. Sheet piles can be made of wood, pre-cast concrete, or steel. Wood is an ineffective water barrier, however, and concrete is used primarily where great strength is required. Steel is the most effective in terms of groundwater cut-off and cost, and [illegible] discussed here.

5.3.3.1 Applications and Limitations

Steel sheet piling can be employed as a groundwater barrier much like the others discussed in this chapter. Because of costs and unpredictable wall integrity, however, it is seldom used except for temporary dewatering for other construction, or as erosion protection where some other barrier, such as a slurry wall, intersects flowing surface water.

One of the largest drawbacks of sheet piling, or any other barrier technology requiring pile driving, is the problem caused by rocky soils. Damage to or deflection of the piles is likely to render any such wall ineffective as a groundwater barrier.

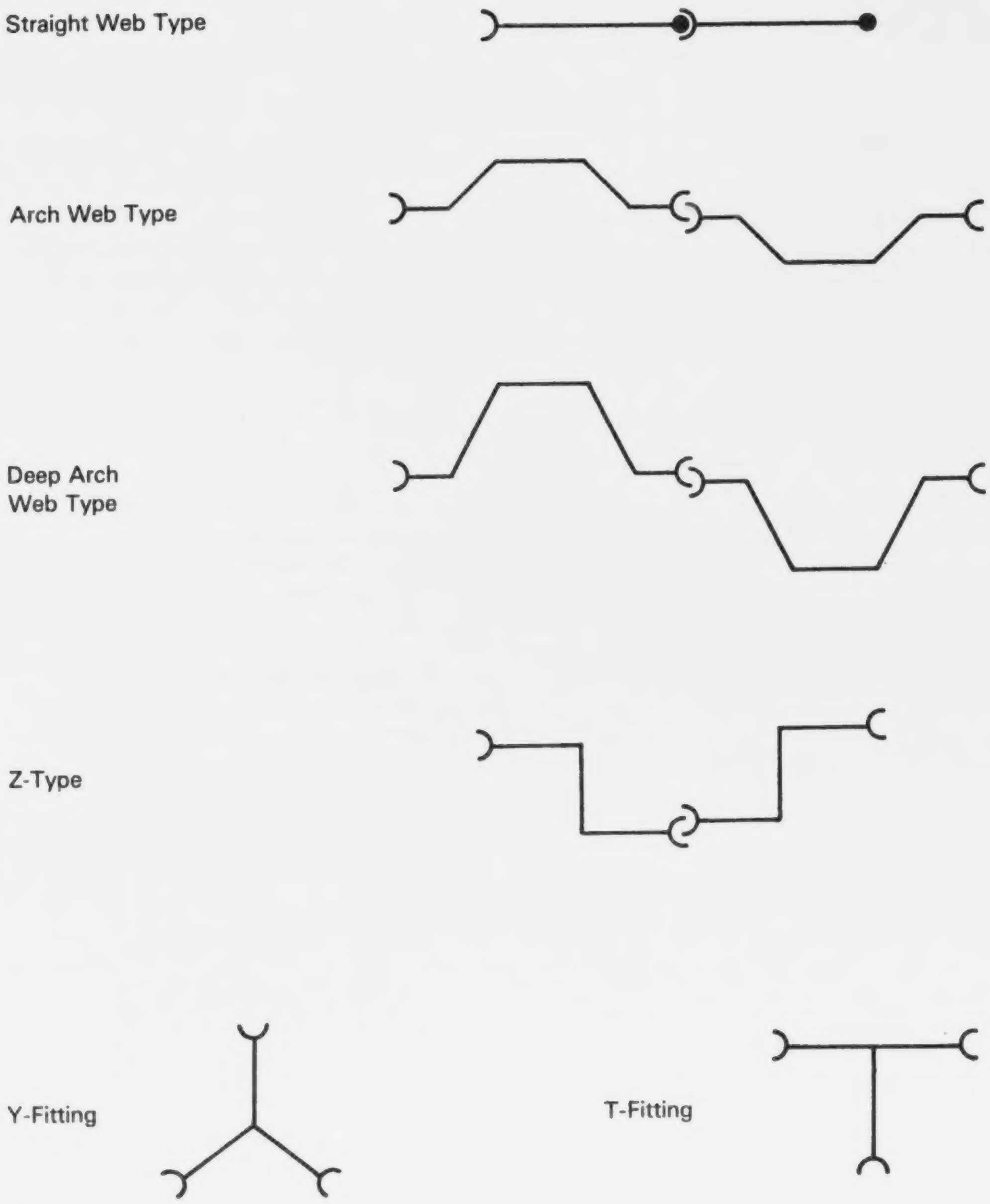
5.3.3.2 Design and Construction Considerations

The primary design parameters for any barrier are permeability and dimensions. Dimensional requirements are based on site characteristics and are straightforward. Depth limitations are governed by the soil material at the site. Design factors for ultimate permeability of the cut-off are more complicated and must assume some factor to account for leakage through the interlocking joints.

Typical shapes for steel sheet piling are shown in Figure 5-36.

For construction of a sheet piling cut-off, the pilings are assembled at their edge interlocks before they are driven into the ground. This is to ensure that earth materials and added pressures will not prevent a good lock

FIGURE 5-36.
SOME STEEL PILING SHAPES AND INTERLOCKS



Source: Ueguhardt et al., 1962

between piles. The piles are then driven a few feet at a time over the entire length of the wall. This process is repeated until the piles are all driven to the desired depth.

The sheet piling is forced into place by a drop hammer or a vibratory hammer. Heavy equipment is desirable for fast driving and to prevent damage to the piles. Lightweight equipment can distort the top edge of the pile and slow the driving (ARMCO, Inc., Baltimore, MD, personal communication, 1980). Often, a cap block or driving head is placed on the top edge to prevent the driving equipment from damaging the piles.

When first placed in the ground, sheet piling cut-offs are quite permeable. The edge interlocks, which are necessarily loose to facilitate placement, allow water easy passage. With time, however, fine soil particles are washed into the seams and water cut-off is effected. The time required for this sealing to take place depends on the rate of groundwater flow and the texture of the soil involved. In very coarse, sandy soils, the wall may never seal. In such cases, it is possible to grout the piling seams, but this is a costly procedure.

5.3.3.3 Operation, Maintenance, and Monitoring

Steel sheet piling cut-offs require little maintenance. In corrosive soils, galvanized or polymer-coated piles can prolong the service life of cut-off, as will cathodic protection, but these are preconstruction measures. Monitoring of sheet piling cut-offs parallels that for other barriers, and involves monitoring head levels and groundwater quality on either side of the barrier to determine if it is functioning as designed.

5.3.3.4 Technology Selection/Evaluation

The performance life of a sheet piling wall can be between 7 and 40 years, depending on the condition of the soil in which the wall is installed. Sheet piling walls have been installed in various types of soils ranging from well-drained sand to impervious clay, with soil resistivities ranging from 300 ohm/cm to 50,000 ohm/cm, and with soil pH ranging from 2.3 to 8.6. Inspections of these installed walls did not reveal any significant deterioration of the structure due to soil corrosion (USEPA, 1978). Additional protection of the sheet piling wall against corrosion can be achieved by using hot-dip galvanized or polymer-coated sheet. Cathodic protection has also been suggested for submerged piling (USEPA, 1978).

Steel sheet piles should not be considered for use in very rocky soils. Even if enough force can be exerted to push the piles around or through cobbles and boulders, the damage to the piles would be likely to render the wall ineffective.

5.3.3.5 Costs

Costs for installed steel sheet piling will vary with depth, total length, type of pile (coated or uncoated), and relative ease of installation. Average costs range from approximately \$6.50 per square foot up to approximately \$16.00 per square foot (Godfrey, 1984a and McMahon, 1984a).

5.3.4 Bottom Sealing

Bottom sealing refers to techniques used to place a horizontal barrier beneath an existing site to act as a floor and prevent downward migration of contaminants. Most of these techniques involve variations of grouting or other construction support techniques, and no documented application to a hazardous waste site was found in the literature.

5.3.4.1 Grouting

Emplacement of a bottom seal by grouting involves drilling through the site, or directional drilling from the site perimeter, and injecting grout to form a horizontal or curved barrier. One such technique, jet grouting, involves drilling a pattern of holes across the site to the intended barrier depth. A special jet nozzle is lowered and a high pressure stream of air and water erodes the soil. By turning the nozzle through a complete rotation, a flat, circular cavity is formed. The cavity is then grouted with intersecting grouted masses forming the barrier. The directional drilling method is very similar to curtain grouting except that it is performed in slanted rather than vertical boreholes.

Because these techniques are developmental, no detailed analysis of applications, limitations, design, or construction considerations is possible. No cost data, other than the material costs shown in Table 5-15, are available.

5.3.4.2 Block Displacement

Block displacement is an experimental technique for isolating and raising a contaminated block of earth. By this technique, a perimeter barrier is constructed by slurry trenching or grouting. Grout is then injected into specially notched holes bored through the site. Continued grout or slurry pumping causes displacement of the block of earth isolated by the perimeter barrier and forms a bottom seal beneath the block.

This technique has been laboratory tested and field demonstrated at a nonhazardous site (Brunsing and Henderson, 1984). It has yet to be attempted at an actual hazardous waste site and the technique is still being refined. As such, no detailed analysis for its use in waste site remediation is possible, and no cost data are available.

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SECTION 6

GAS CONTROL

The disposal of solid and hazardous waste by landfilling employs engineering principles and construction methods to confine waste to the smallest area practical, compact the waste into the lowest volume possible, and cover the waste with layers of soil to limit exposure of the materials to the environment. This method inadvertently creates conditions in which gases are produced, vented to the atmosphere, and migrate laterally through soil, utilities, and other pathways to outlying areas. Serious accidents resulting in injury, loss of life, and extensive property damage can occur where landfill conditions favor gas migration (Emcon, 1981). Degradation of air quality has resulted from the venting of gases to the atmosphere.

Organic matter within landfills is transformed into a variety of simpler organic materials and byproduct gases by the action of microorganisms that are abundant in refuse. The major components of landfill-generated gas are carbon dioxide and methane; however, lesser amounts of oxygen, nitrogen, and hydrogen sulfide are frequently produced. The refuse experiences first aerobic (oxygen-abundant), then anaerobic (oxygen-deficient) conditions, and the gas composition varies through the transition (Emcon, 1981).

The period of gas generation from a landfill may range from a few years to hundreds of years. The active gas production life is dependent on site-specific conditions. The rate of gas production is dependent on the levels of oxygen present, refuse moisture content, environmental pH, temperature, and refuse composition. Gas production is stimulated by a high percentage of biodegradable materials such as food and garden wastes, paper, textiles, and wood, and can be inhibited by the presence of waste materials that are toxic to the gas-producing microorganisms. The major component of landfill-generated gas is methane, which is odorless, colorless, lighter than air, and combustible. The high combustibility of methane makes it a potential hazard in landfill environments. In concentrations between 5 and 15 percent by volume in air, methane is flammable at atmospheric pressure and ordinary temperatures (Emcon, 1981).

Gases may be formed in landfills by microbiological degradation of organic matter and/or by volatilization of organic liquids. Organic gases can also be present in landfill gas by volatilization of volatile organic liquids (solvents, fuels, etc.) within the landfill or by biological decomposition of mixtures of organic matter and organic liquids. Landfilled organic liquids can inadvertently be present in domestic refuse, be illegally deposited in sanitary landfills, or be placed free-flowing or in containers in industrial and hazardous waste landfills.

The relative ease with which landfill gas is capable of migrating through soil, often to significant distances, exacerbates the hazards of explosion and exposure. Movement of the gas will occur in sand, silt, or clay soils as long as there are connected voids. However, the rate of movement decreases as pore size decreases; therefore, movement will be greatest through highly permeable sand or gravel and least in clay soils. Utility and drainage corridors often provide pathways for methane migration (Emcon, 1981).

In addition to methane-related accidents, migrating gas may result in other adverse effects such as damage to vegetation (depending on plant sensitivity to carbon dioxide and methane, oxygen depletion, and elevated temperatures), nuisance of malodors (carbon dioxide and methane have no odor; malodors may result from volatile organic gases of decomposition or, occasionally, hydrogen sulfide) (Emcon, 1981), and acute and chronic health effects resulting from exposure to toxic gases.

The gases originating within landfills vent to the atmosphere by vertical migration and/or lateral migration. The majority of the gas produced by a landfill normally vents through the cover material. However, if this vertical path is sealed by frost, rain-saturated cover soil, pavement, or "capping" with a clay or synthetic liner, there is a greater tendency toward lateral migration. In general, a landfill constructed in a sand-gravel environment experiences greater lateral movement of gases than one in a clay environment. These concepts are shown in Figure 6-1. Since gas migration and venting can result in significant hazard, special control systems have been developed to alleviate these problems (Emcon, 1981). The following three categories of gas control systems are described in this chapter:

- Passive perimeter gas control systems
- Active perimeter gas control systems
- Active interior gas collection/recovery systems.

6.1 Passive Perimeter Gas Control Systems

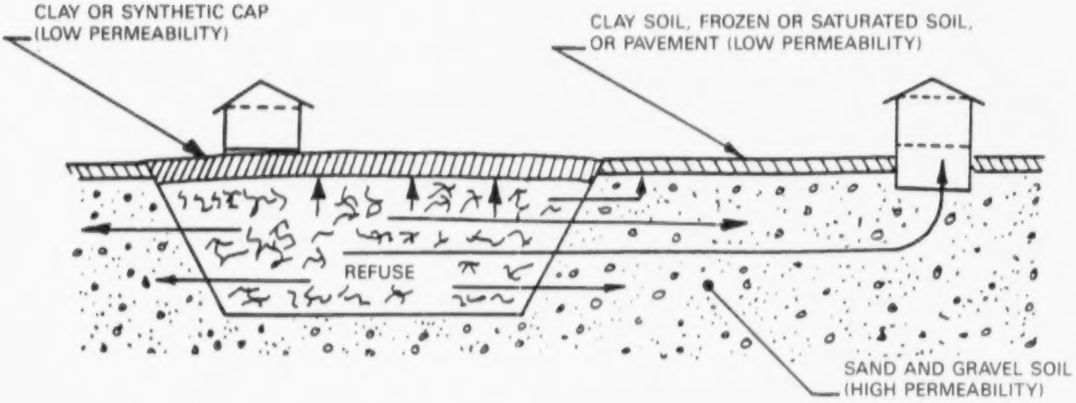
6.1.1 General Description

Subsurface migration of landfill-generated gases beyond the landfill property line (or other appropriate limit) may be prevented through the use of passive gas control systems, i.e., systems that control gas movement by altering the paths of flow without the use of mechanical components. Passive systems may be further categorized as high-permeability or low-permeability systems.

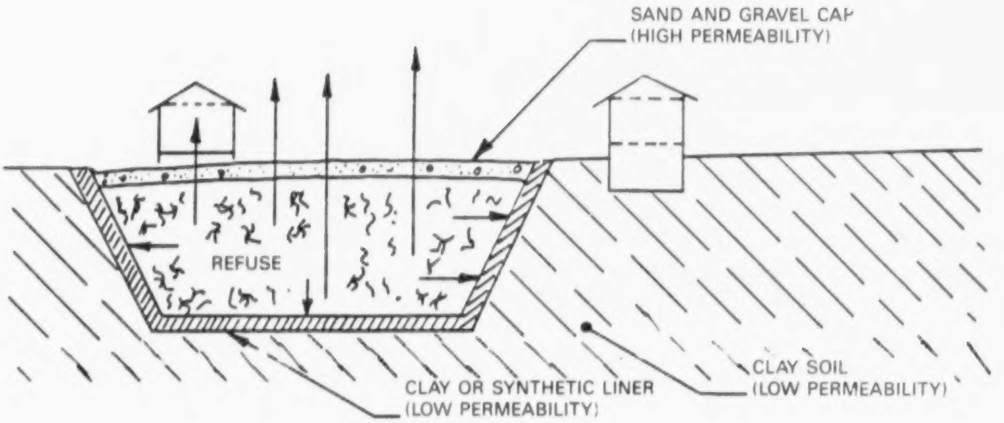
High-permeability systems entail the installation of highly permeable (relative to the surrounding soil) trenches or wells between the landfill and

FIGURE 6-1. PATHWAYS OF GAS MIGRATION

EXTENSIVE LATERAL MIGRATION



EXTENSIVE VERTICAL



Source: Emcon, 1981

the area to be protected as shown in Figure 6-2. Since the permeable material offers conditions more conducive to gas flow than the surrounding soil, paths of flow to points of controlled release are established. High-permeability systems generally take the form of trenches or wells excavated outside of the landfill limit and backfilled with a highly permeable medium such as a coarse crushed stone. As well spacing decreases, the design and function of well vents approaches that of trench vents (SCS, 1980).

Low-permeability systems, as shown in Figure 6-3, effectively block gas flow into areas of concern by the use of barriers (such as synthetic membranes or natural clays) between the landfill and the area to be protected. With low-permeability systems, gases are not collected and therefore cannot be conveyed to a point of controlled release or treatment. The purpose of the system is to prevent or reduce gas migration into areas that are to be protected. These two concepts (high- and low-permeability) of passive gas control are often combined in the same system to provide controlled venting of gases and blockage of available paths for gas migration (SCS, 1980).

Low-permeability passive control systems also attempt to check gas migration by altering the paths of convective flow, but also prevent or impede diffuse flow by blocking available paths. By placing a relatively impermeable material in paths of gas flow, the pressure gradient across the barrier is "flattened" due to the high resistance. Gases will preferentially flow along paths of steeper pressure gradients in order to vent to the atmosphere. These new paths will be either through the ground surface between the barrier and the landfill or through the landfill surface. Diffuse gas flow through the barrier is minimal since diffuse flow also requires paths for gas movement. A barrier would effectively block the paths and contain diffuse flow (SCS, 1980).

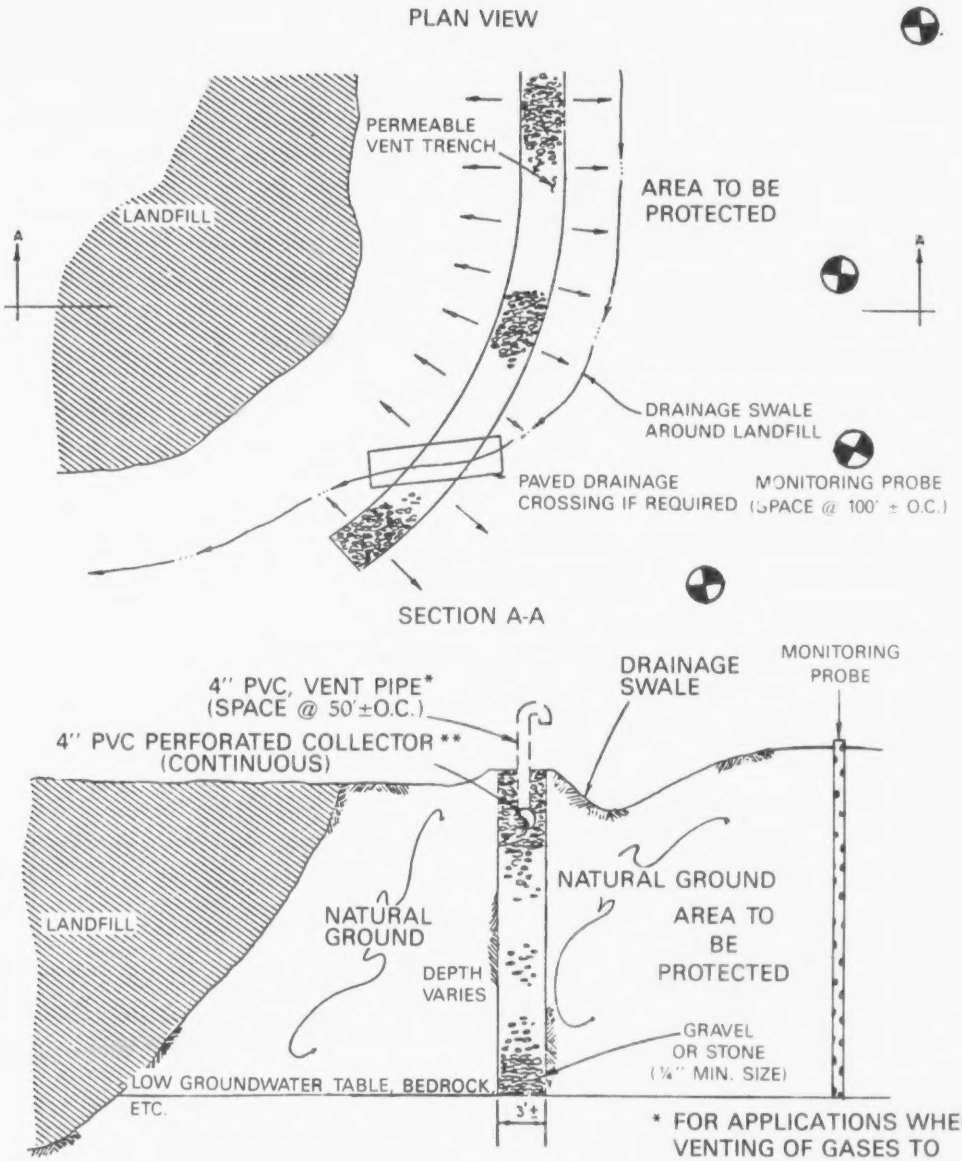
6.1.2 Applications/Limitations

Passive gas control systems can be used at virtually any site where there is capability to trench or drill an excavation to at least the same depth as the landfill. Limiting factors could include the presence of a perched water table or rock strata. Passive vents should generally be expected to be less effective in areas of high rainfall or prolonged freezing temperatures.

6.1.3 Design Considerations

A schematic diagram of a high-permeability trench is shown in Figure 6-2. The width of the trench is dictated by the characteristics of available excavation equipment, the slope stability of the soil being excavated, and the depth of the trench. Minimum trench widths of 3 feet are often specified in order to ensure an open trench over the full depth. The depth of the trench is dictated by local site conditions. In general, the trench should extend from the ground surface to a relatively impermeable stratum of unfractured

FIGURE 6-2.
PASSIVE GAS CONTROL USING A PERMEABLE TRENCH

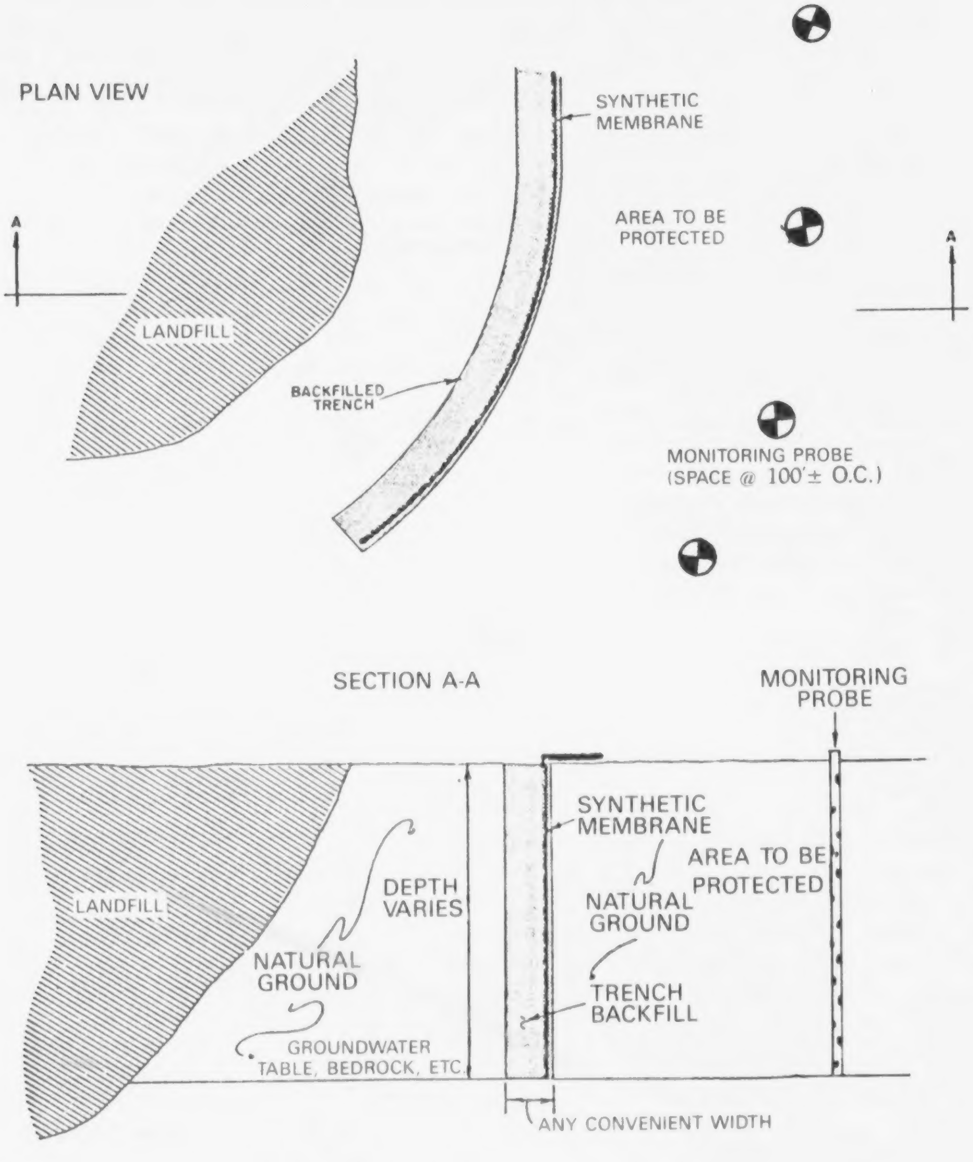

PLAN VIEW
PERMEABLE VENT TRENCH
LANDFILL
AREA TO BE PROTECTED
A
A
DRAINAGE SWALE AROUND LANDFILL
PAVED DRAINAGE CROSSING IF REQUIRED
MONITORING PROBE (SPACE @ 100' ± O.C.)
SECTION A-A
4" PVC, VENT PIPE* (SPACE @ 50'±O.C.)
4" PVC PERFORATED COLLECTOR ** (CONTINUOUS)
DRAINAGE SWALE
MONITORING PROBE
LANDFILL
NATURAL GROUND
NATURAL GROUND
AREA TO BE PROTECTED
DEPTH VARIES
GRAVEL OR STONE (¼" MIN. SIZE)
LOW GROUNDWATER TABLE, BEDROCK, ETC.
3'±

* FOR APPLICATIONS WHERE VENTING OF GASES TO ATMOSPHERE IS ACCEPTABLE.

** COLLECTOR CAN BE USED TO CONVEY GASES TO A TREATMENT SYSTEM.

Source: SCS, 1980

FIGURE 6-3.
PASSIVE GAS CONTROL SYNTHETIC MEMBRANE



Source: SCS, 1980

bedrock or clay or to the lowest groundwater table level. In some applications, the trench need not be as deep, so long as it extends to a sufficient depth to intercept all possible avenues of gas migration. This depth is a function of the landfill depth and the geology in the vicinity of the landfill (SCS, 1980). The logistics of excavating open trenches (i.e., bracing or sloping trench walls) can constrain the use of passive venting trenches to relatively shallow depths of 30 feet and less.

Crushed stone or river gravel is normally used as the permeable-medium trench backfill. Stone sizes greater than 1/4-inch are recommended; fine material should not be used. As shown in Figure 6-2, horizontal perforated pipe and vertical solid-wall riser pipes are often used to ensure that paths of gas flow to the atmosphere remain open in the event that the top of the trench becomes blocked by ice, snow, vegetation, etc. The ground surface should be graded to drain away from the trench to prevent the washing of soil into the voids of the stone. If drainage swales must cross the trench, they should be installed using enclosed conduits or paved channels.

Low-permeability systems would normally be installed in a trench situated and excavated in a manner similar to that used in installing a passive vent trench. The width and depth requirements would essentially be the same. However, in lieu of the highly permeable stone backfill, low-permeability material would be placed. Synthetic membranes are normally used as barriers in this application (SCS, 1980).

The synthetic membrane is normally draped over the wall of the trench furthest from the landfill. A schematic diagram of a synthetic membrane barrier is shown in Figure 6-3. The membrane must be continuous in order to be effective and many splices may be required, depending upon the depth and length of the trench. The membrane should completely over the wall of the trench and the end is often temporarily laid on the ground surface at the top of the trench to hold it in place. It is necessary that extreme caution be taken during the installation to prevent tearing or puncturing of the membrane. This would include "dressing" the trench wall to remove protruding objects such as roots or jagged rock fragments. Once the membrane has been placed and secured at ground level, the trench is carefully backfilled with any material that would not puncture the membrane or settle appreciably after being placed. Native soil, sand, or rounded gravel is often used for this purpose. Once the entire trench has been backfilled, excess membrane above the ground surface may be trimmed (SCS, 1980).

Soil-bentonite slurry cut-off walls have been used as gas migration control barriers. The saturated soil-bentonite slurry backfill serves the same function as the compacted clay or membrane barrier. However, the effectiveness of slurry walls as barriers depends on the slurry remaining saturated, a condition that is assured at depths below the groundwater table. Special provisions, such as the application of water, may be required to assure continuous saturation of the slurry and thus its effectiveness as a barrier. Information pertaining to the effectiveness of slurry walls over a range of applications is limited; this technique is not conventional and is not widely recognized for gas migration control. Information pertaining to slurry walls for groundwater control and protection is provided in Section 5-3.

High-permeability and low-permeability passive control concepts are often combined into a single passive gas control system. A synthetic membrane is installed along the trench wall as described above under low-permeability systems. From that point, the trench is backfilled as described under high-permeability systems, except that care must be taken to avert damaging the membrane. Such a system relies upon the high-permeability material to control both convective and diffuse flow. Since a relatively "steep" pressure gradient (gravel) is followed immediately by a material extremely resistant to gas flow (membrane), landfill gases are further encouraged to flow through the gravel and vent piping to the atmosphere. In addition, the combination of the two concepts provides a degree of redundancy, affording protection in the event that one portion of the system is inadequately designed or improperly installed (SCS, 1980).

6.1.4 Construction/Implementation Considerations

Trenches excavated for passive gas control systems are normally cut with backhoes, although other conventional trenching equipment allowing for adequate depth and width could be used. Rounded gravel or crushed stone, washed of fines, should be used for venting trench backfill material, and only rounded gravel should be used as a permeable medium in conjunction with a synthetic membrane liner to avert tearing or puncturing the membrane. Virtually any pipe material may be used for perforated and riser pipe, although 3- or 4-inch PVC pipe is customarily used. Polyvinyl chloride (PVC), polyethylene (PE), chlorinated polyethylene (CPE), Hypalon®, and other materials have been utilized for impermeable synthetic membranes. A minimum thickness of 20 mils is recommended. Lap joints are cemented or heatwelded and may be made at the factory or in the field (SCS, 1980).

During trench excavation for passive systems, care should be taken to ensure that workers are not overcome by venting gases or exposed to explosion hazards. Open flames and smoking should be prohibited in the work area. Regular monitoring of methane, oxygen, hydrogen sulfide, and other gases of concern should be conducted. Depending on soil characteristics and trench depth, sloping of trench walls may be required to avoid instability; alternatively, shoring and bracing can be used to support trench walls. This does not adversely affect the installation, although additional backfill material is required as a result. During the installation of synthetic membrane barriers, extreme care must be taken to ensure that lap joints are properly sealed and that tears and punctures are averted in the process of placing the membrane and backfilling the trench (SCS, 1980).

6.1.5 Operation, Maintenance, and Monitoring

Passive gas control systems, by definition, alter the paths of gas flow without the use of mechanical components. As such, the systems are essentially self-operating. Vent pipes, drainage patterns, and general conditions in the vicinity of the systems should be occasionally (e.g., monthly) inspected to identify the need for repairs or other maintenance.

Monitoring the effectiveness of passive gas control systems normally consists of periodic sampling of subsurface gases from probes installed in the area being protected. Conceptual probe locations are shown in Figures 6-2 and 6-3 and a typical probe design is shown in Figure 6-4.

Installation of a gas monitoring probe requires drilling a hole in the soil to a depth over which monitoring is desired. The probe pipe is perforated except for the upper several feet. The probe is installed in the drilled hole, and the hole is backfilled with permeable material (sand or pea gravel) to a height above the perforations. The remainder of the hole is backfilled with soil to act as a seal against the intrusion of air. Seals are sometimes used to keep soil from entering the permeable material. A gas sample can then be withdrawn from the probe at the surface (Emcon, 1981).

A thin-walled PVC (polyvinyl chloride) pipe is commonly used for the probe casing. To obtain a representative gas sample, the probe must be purged to ensure that gas from the soil is being withdrawn. Since methane is lighter than air and carbon dioxide is heavier, some stratification of the gas is frequently observed in the probe (Emcon, 1981).

Gas samples can be collected in bottles for laboratory analysis; however, readings taken with portable meters or organic vapor analyzers are more commmon. Generally, the presence of methane or other combustible gas or fume can be detected with devices that provide scales of measure in "percent combustible by volume" and/or "percent of LEL" (lower explosive limit). These devices are commonly used by fire departments, natural gas and sewer utilities, and the mining and refining industry. Organic vapor analyzers and gas chromatographs provide further identification and quantification of mixtures of gases.

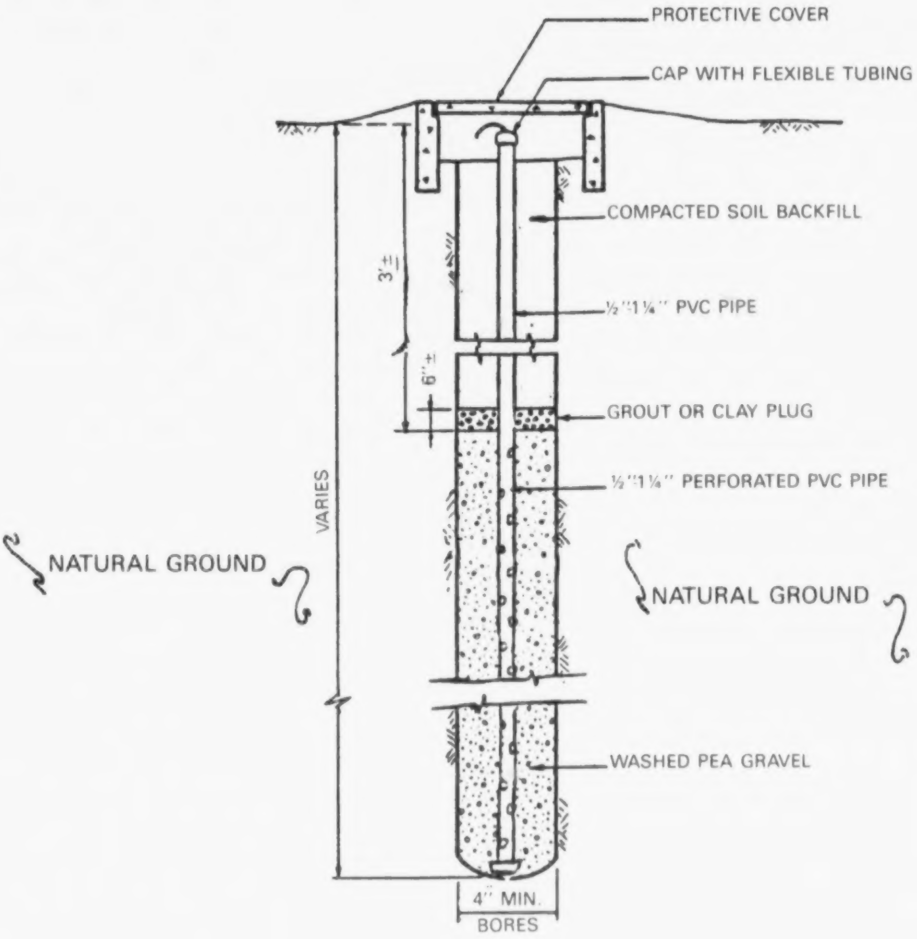
The effectiveness of gas control can also be monitored by automatic detectors that sense the presence of combustible and/or toxic gases and sound alarms or trigger other responses. The detectors are placed in utility vaults, living and working areas, ventilation systems, and subsurface soils. The detectors are sensitive to changes in temperature and humidity, and require regular calibration and periodic replacement.

The presence of combustible and/or toxic gases at a monitoring point is an indication that the gas migration control system is not affording the protection that is intended and the flaw in the system should be identified and corrected.

6.1.6 Technology Selection/Evaluation

Subsurface pressure is a driving force that causes landfill gas to flow from the landfill to the atmosphere. High-permeability systems attempt to check gas migration by altering the paths of convective (pressure) flow. In

FIGURE 6-4.
TYPICAL GAS MONITORING PROBE



Source: EMCON, 1980

providing a highly permeable path for gas flow, the pressure gradient is "steepened" (by reducing the length of the path of flow to the atmosphere). Gases tend to flow in the direction of the steepest pressure gradient, often referred to as the "path of least resistance." Passive vents, however, do little to control diffuse gas flow. Gases moving under diffuse flow tend to move randomly in all directions from the point of generation or release. Although the potential for gas migration is usually much lower under diffuse flow than under convective flow, many passive vents have been ineffective or limited in effectiveness by diffuse gas flow passing through the permeable medium and continuing migration beyond the vent.

High-permeability gas control systems have functioned adequately in many applications; however, there appear to be no clear patterns which dictate success or failure of the systems (SCS, 1980). While passive vents may perform effectively at some sites, the method cannot be considered to be reliable for landfill gas migration control because of the inability of vents to control diffuse flow. Numerous passive well venting systems have been converted to active systems (see Section 6.2) because of poor or unreliable performance. Low-permeability systems block diffuse flow and are highly reliable when properly designed and installed. Combined low- and high-permeability systems offer the highest level of effectivenesss and reliability of passive gas control. Since passive systems require virtually no operation and maintenance, their performance and reliability are not limited by manual upkeep or continuous operation of mechanical components.

Passive gas control systems can be implemented with relatively conventional construction equipment, labor, and materials. Handling and placement of synthetic liners requires specialized equipment and labor. Slope stability of soils being excavated is an important consideration. Laying back of trench walls requires additional excavation and working area, and placement of shoring or bracing in the trench requires additional labor and materials. Workers should not enter excavated areas without adequate respiratory and other appropriate protection and rescue provisions. No flames or smoking should be allowed in the vicinity of open excavations.

Passive control systems require relatively little time to implement. A single crew can complete up to several hundred feet of perimeter per day. Gas control is affected as soon as the vent is complete, although a short time may be required for gas that has already migrated to dissipate.

6.1.7 Costs

Because the gas control systems described herein are "passive" in concept, virtually no operating or maintenance costs should be incurred. It is recommended, however, that periodic visual inspections be made in the area of the system to determine whether local activities may have interfered with the system's effectiveness. Also, subsurface gas should be periodically monitored in the area being protected to ensure that the systems are performing their intended functions.

Costs associated with treatment of collected gases will also contribute to the total cost of gas control. Treatment of gaseous waste streams is addressed in Section 10.2.

The following is an example of an estimate of the costs of a passive perimeter gas migration control system:

Scenario:

- Closed landfill containing primarily domestic refuse, free liquid organic wastes are mixed with solid waste.
- Average landfill depth is 40 feet.
- Landfill perimeter is 2,300 feet.
- Continuous low-permeability clay exists at depths of 25 feet and below and underlies the bottom of the landfill.

Required:

- Estimate the capital and O&M costs of a passive combined low-permeability and high-permeability perimeter gas migration control system.

Assumptions:

- Barrier trench should extend 30 feet below the ground surface.
- By off-setting the trench from the edge of landfill, the length of the trench will be 2,500 feet.
- The width of the trench will be 3 feet.
- Collector piping will be installed over the full length of the trench.
- Vent pipes will be 10 feet long and will be spaced at 50 feet.
- The trench will be lined with a synthetic membrand and backfilled with washed gravel.
- Gases will be vented to the atmosphere.
- Monitoring probes will be spaced at 100 feet and will be 30 feet deep.
- Probes will be monitored quarterly with a portable meter.

Estimates of Quantities:

- Trench excavation and backfill: $30' \times 2,500' \times 3' \div 27\ ft^3/yd =$ 8,333 cubic yards

- Horizontal piping: 2,500 linear feet

- Vertical piping: 2,500' ÷ 50' x 10' = 500 vertical feet

- Synthetic membrane: 2,500' x 30' = 75,000 sq. ft.

- Monitoring probes: 2,500' ÷ 100' x 30' = 750 vertical feet

- Monitoring: 2,500' ÷ 100' x 4 visits/year = 100 visits/year

Estimate of Capital Costs:

Item	Quantity	Unit Cost Low	Unit Cost High	Item Cost Low	Item Cost High
Trench excavation	8,333 yd^3	$ 2	$ 4	$ 16,666	$ 33,332
Dispose of excavated material	8,333 yd^3	$ 2	$ 4	$ 16,666	$ 33,332
Gravel backfill	8,333 yd^3	#1	$18	$ 99,996	$149,994
Horizontal piping	2,500 l.f.	$ 4	$ 6	$ 10,000	$ 15,000
Verticalpiping	500 v.f.	$ 4	$ 6	$ 2,000	$ 3,000
Synthetic membrane	75,000 sq. ft.	$ 2	$ 4	$150,000	$300,000
Monitoring probes	750 v.f.	$10	$15	$ 7,500	$ 11,250
Total Capital Costs				$302,828	$545,908

Estimate of Annual O&M Costs:

Item	Quantity	Unit Cost Low	Unit Cost High	Item Cost Low	Item Cost High
Monitoring with portable meter	100 visits	$10	$15	$ 1,000	$ 1,500
Total Annual O&M Costs				$ 1,000	$ 1,500

Since the lengths, depths, and widths of passive system trenches vary, the unit cost per linear foot of landfill border will be totally site-specific. The capital costs in Table 6-1 are given in units that can be readily determined for a given site, with some judgment on the part of the estimator.

TABLE 6-1.
1985 UNIT COSTS FOR COMPONENTS OF PASSIVE LANDFILL GAS CONTROL SYSTEMS

Item	Unit	Cost Per Unit
Trench excavation by backhoe	yd^3	\$2-4
Dispose of excavated material on site	yd^3	\$2-4
Crushed stone or gravel backfill, in place	yd^3	\$12-18
Bank sand backfill, in place	yd^3	\$6-9
Horizontal and vertical piping	Linear ft	\$4-6
Synthetic membrane, in place	ft^2	\$2-4
Monitoring with portable meter	Each visit	\$10-15
Monitoring probe (drilling, pipe, fittings, backfill, etc.), in place	Vertical ft	\$10-15

Source: SCS, 1980; SCS, 1985; Godfrey, 1984.

6.2 Active Perimeter Gas Control Systems

6.2.1 General Description

Off-site landfill gas migration can also be controlled through the use of "active" control systems that alter pressure gradients and paths of gas movement by mechanical means. These systems normally consist of three or four major components (SCS, 1980):

- Gas extraction wells
- Gas collection headers
- Vacuum blowers or compressors

- Gas treatment or utilization systems.

A schematic diagram of an active system is shown in Figure 6-5. Centrifugal blowers create vacuum through the collection headers and wells to the wastes and ground surrounding the wells. A pressure gradient is thereby established, inducing flow from the landfill (which is normally under positive pressure) to the blower (creating a negative, or vacuum, pressure). Subsurface gases flow in the direction of decreasing pressure gradient (through the wells, the header, and the blower) and are released directly to the atmosphere, treated and released to the atmosphere, or recovered for use as fuel (SCS, 1980). Treatment of gaseous waste streams is addressed in Section 10.2.

6.2.2 Applications/Limitations

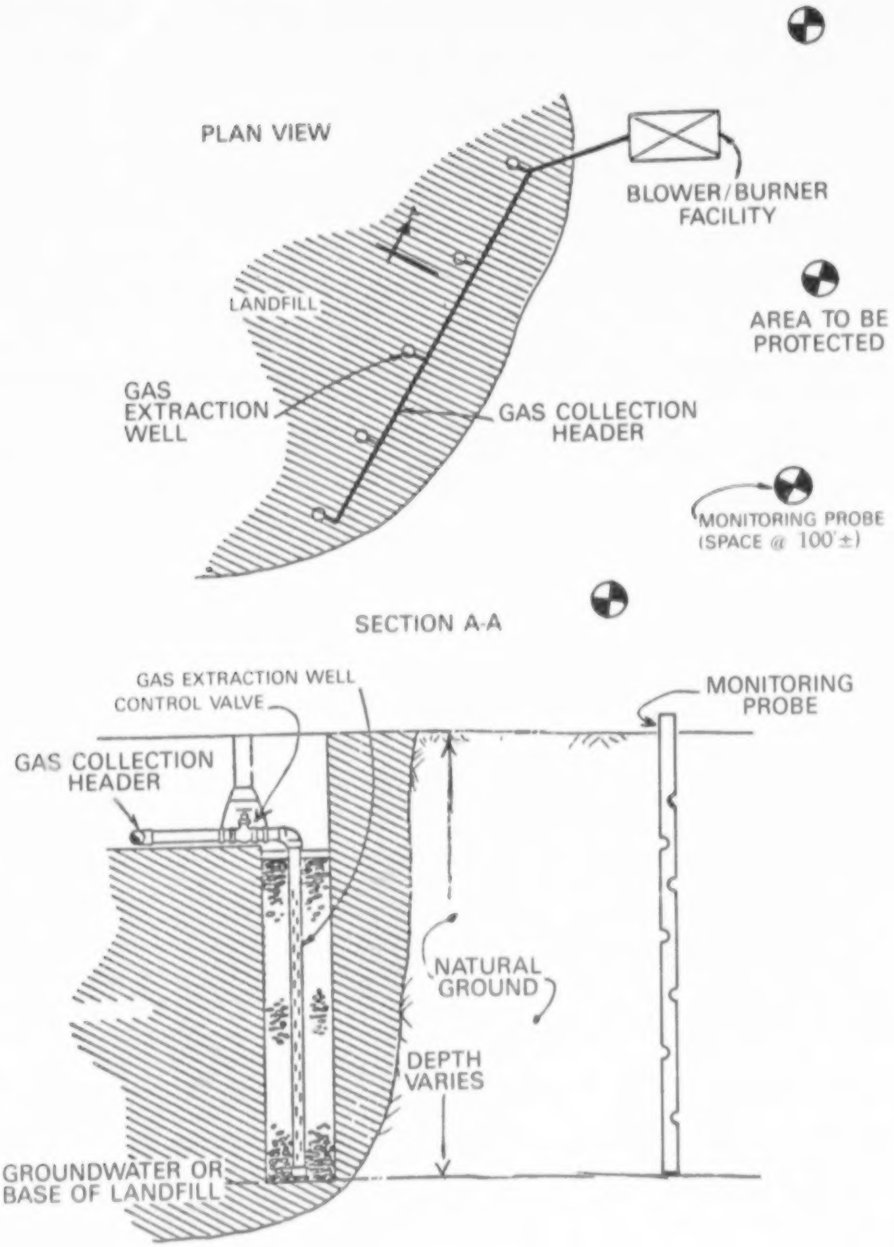
Active perimeter gas control systems can be used at virtually any site where there is capability to drill an excavation through landfilled material to the required depth. Limiting factors could include the presence of free-standing leachate (i.e., saturation) or impenetrable materials within the landfill. Active systems are not sensitive to the freezing or saturation of surface or cover soils.

6.2.3 Design Considerations

Gas extraction wells may be installed either in refuse fill or in soil outside of the limit of fill. The wells in the schematic diagram in Figure 6-5 are shown in the refuse fill. Wells normally consist of a drilled excavation 12 to 36 inches in diameter which is backfilled with one-inch or larger crushed stone and 2- to 6-inch piping, which is perforated in the area where gas is to be collected and solid in the upper portions. A schematic detail of a gas extraction well is shown in Figure 6-6. Solid-wall pipe is used and a concrete or clay seal is provided in the upper portion of the well to minimize infiltration of atmospheric air into the system. A valve is provided on the lateral connection of each well to allow regulation of flow and balancing of systems consisting of multiple wells. A monitoring port is provided for measuring velocity, pressure, and gas composition. Wells are normally drilled to the depth of the seasonally low groundwater table or to the base of the landfill, whichever is the lesser depth. However, geologic conditions and/or landfill characteristics may warrant deeper installations.

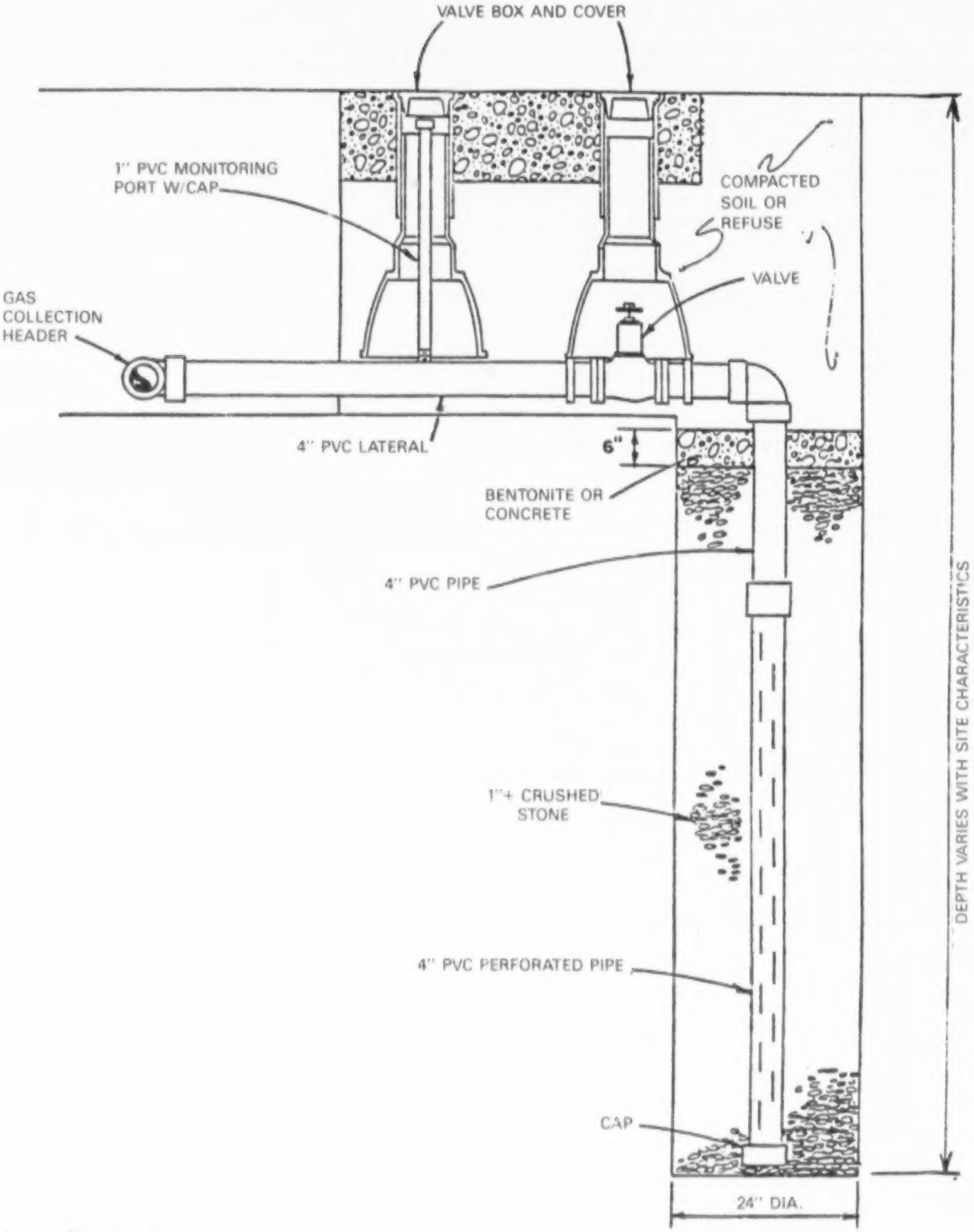
Well spacing is a critical factor in the design of the system and requires considerable judgment on the part of the designer. Spacings on the order of 100 feet are commonly used, however, the appropriate spacing for a given site will depend upon the depth of the landfill, the magnitude of the vacuum applied to the well, and the rate of gas withdrawal (SCS, 1980). Where extraction wells are installed within landfilled material, additional considerations are the type of waste (solid, sludge; domestic, industrial; organic, inorganic), the moisture content of the waste, and the degree of

FIGURE 6-5.
ACTIVE GAS EXTRACTION



Source: SCS. 1980

FIGURE 6-6.
GAS EXTRACTION WELL



Source: SCS, 1980

compaction of the waste. Where wells are installed in soil outside of the landfill, the grain size distribution, moisture content, stratigraphy, and permeability of the soil should be considered.

In order to determine appropriate values for design criteria, gas extraction tests should be performed on one or more test wells while monitoring the change in pressure gradient radially from the wells. Parameters that should be monitored during the tests are:

- Gas extraction flow rates
- Subsurface negative (vacuum) pressures at various depths and distances from the well(s)
- Negative pressures within the well.

The data should be collected under several flow conditions. Based on the data, several combinations of flow rates, vacuum pressure applied at the well, and effective "radius of influence" of the wells can be established. The radius of influence is the distance from the gas extraction well beyond which gases are not induced to flow toward the well and is usually the distance corresponding to a "cut-off" (close to zero) vacuum pressure. In design, the zones of influence of adjacent wells are slightly "overlapped" to establish the well spacing. The most cost-effective combination of flow rate, vacuum pressure, and well spacing is selected for the system design. This will depend on cost, whether gas recovery and use are anticipated, and optimization of gas quality and quantity (SCS, 1980).

With the basic design criteria having been determined as described above, appropriate well locations may be established to suit the configuration of the landfill perimeter. The header system is then laid out to convey vacuum from a blower (or compressor) to the wells, and thus induce the flow of extracted gas from the wells to the blower/burner facility. The configuration of the header system also depends on the perimeter configuration, and the header pipe size(s) is determined through standard flow/pressure calculations (SCS, 1980). Header pipes are sized on the basis of flow rate and permissible pressure loss in the header line. Flow equations analogous to those used for design of air and water distribution systems can be used to correlate pressure loss with gas flow rate, pipe length, and pipe material. Using a commonly employed pipe-friction equation, pressure loss is a function of a friction factor, pipe length, the mean gas velocity, and pipe diameter. The friction factor is a function of the relative roughness of the pipe. Methods of solution for the flow of gas in air conditioning and heating networks are in common usage, and standard texts should be consulted for details (Emcon, 1980).

The first step in designing a gas collection header is to estimate gas flow rates from the individual extraction wells. Since preliminary flow rate estimates may be inaccurate, a factor of safety should be used to adjust the flow rate upward (Emcon, 1980). Cumulative gas flow rates along the header line are estimated by summing the individual well flow rates "upstream" from the point under consideration.

Once the flow distribution has been estimated for a proposed header layout, segments of the header pipe can be sized to keep pressure losses within limits of available motor/blower units. Alternative header layouts can be proposed, each requiring different lengths of various sizes of pipe. Since the installation cost per foot is less for smaller pipe sizes than it is for larger sizes, there is a cost trade-off between length and pipe diameter (Emcon, 1980).

Header system piping may be buried, installed on the ground surface, or elevated above the ground. Factors that influence this decision are climate, aesthetics, interfering land uses, potential vandalism, and cost (SCS, 1980).

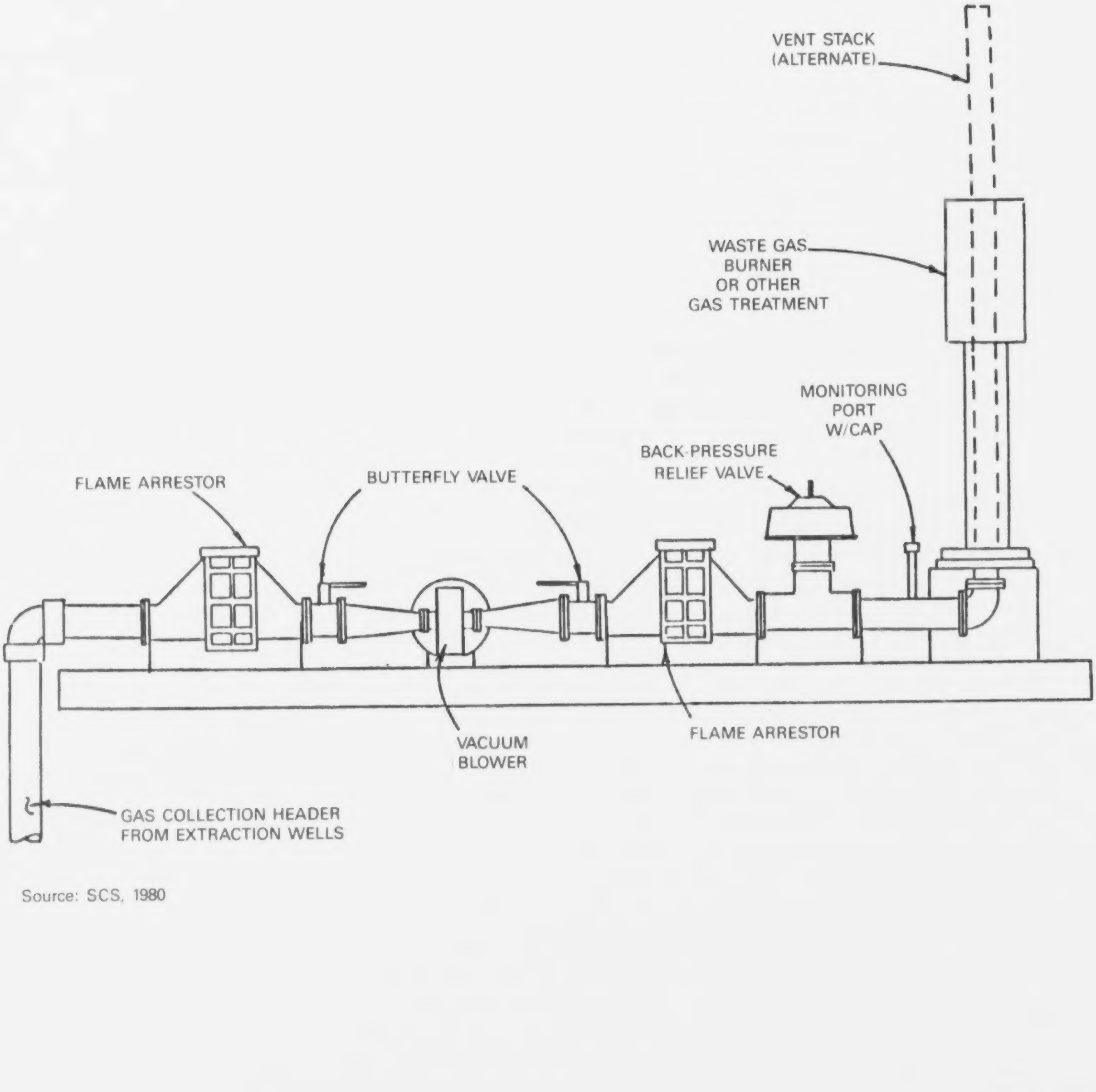
Extracted landfill gas usually has a high moisture content and, as a result, the collection of condensate water in the system components must be considered in the design. Materials that resist chemical attack should be used in collection systems because condensate can be highly corrosive. Condensate water may be effectively drained from the header system by sloping the piping in the direction of flow, when possible, and considering the "drag" of the gas flow against the draining condensate when sloping the piping against the direction of gas flow. Overflow devices (such as traps) should be installed at low points in the header to dispose of condensate and avert blockages of gas flow (SCS, 1980).

The most common practice for handling condensate water is to return it to the landfill as it is collected. However, condensate is potentially a "hazardous" waste under Federal (RCRA) and some state regulations, regardless of whether the site of condensate generation is a hazardous waste or a non-hazardous waste landfill. Condensate that is ignitable, corrosive, reactive, or EP toxic (per RCRA) is considered to be a hazardous waste and cannot be returned to nonhazardous waste or closed hazardous waste landfills. The acceptability of returning condensate to the landfill must be considered on a case-by-case basis. Alternative practices are pretreatment followed by discharge to sewerage systems and removal of condensate from the site for disposal elsewhere, or use as a fuel extender (Paul, 1985).

The extracted gas flows through the header system to the blower/treatment facility. This facility normally consists of a vacuum blower, regulating valves, safety devices, and a waste gas burner (if required). A schematic diagram of a typical blower/treatment facility employing a flare is shown in Figure 6-7. The flame arrestors and backpressure relief valves are safety devices that are intended to prevent injury and damage to equipment in the event of a "flareback" in the system. The butterfly valves are for regulating flow and preventing passive gas flow when the system is not in service. Gas treatment or destruction is necessary for odor control or to prevent the discharge of hazardous gas or vapors to the atmosphere. However, if such emissions are not of concern, direct venting through a vent stack to the atmosphere may be acceptable (SCS, 1980). Treatment of gaseous wastes is addressed in Section 10-2.

Extracted landfill gas is occasionally recovered for use as a high or low btu fuel. Systems designed solely for gas migration control seldom produce gases of sufficient quality to warrant recovery. However, installation of

FIGURE 6-7.
TYPICAL BLOWER/TREATMENT FACILITY



Source: SCS, 1980

combined landfill gas control/recovery systems at larger sites can be feasible where there are nearby potential users of the gas. The combined system is similar in concept to a gas migration control system. The major differences are that wells would be installed in the interior of the landfill, and the design and operating criteria for optimizing gas quality and quantity and minimizing air infiltration become critical. The proximity to the landfill of a user of the gas may limit the feasibility of methane recovery. Technical and economic analyses and control and recovery systems design should be performed by professionals experienced in the field (SCS, 1980).

Active gas control systems consist of several components, all of which require different materials and installation techniques. Table 6-2 summarizes the requirements for the major system components. Specific material selection is the discretion of the designer; however, the materials listed are those that experience has proven to perform satisfactorily. The need for corrosion resistance and flexibility (in anticipation of landfill settlement) are of particular importance in selecting materials and designing system components (SCS, 1980).

6.2.4 Construction/Implementation Considerations

In general, all gas extraction wells should be constructed before any header pipe is installed. This is recommended because wells are often relocated in the field during construction for a variety of reasons, and realignment of header configurations considering the final well locations may be desirable. Blower/burner facility construction may normally begin at any time, since its location is dictated by factors of accessibility. Associated header alignments may be adjusted to accommodate the facility (SCS, 1980).

The presence of large obstacles in landfills can cause refusal during well drilling, therefore, high-torque drill rigs should be employed. The presence of explosive, ignitable, pressurized, or shock-sensitive materials in the landfill could cause injury to workers and damage to equipment during drilling and trench excavation. Materials that are excavated may be hazardous and should be treated as such. As many landfills are known to contain materials that were prohibited by permit, excavation of landfill materials must be conducted with caution and an appreciation of potential consequences.

Care should also be exercised during system installation to keep flame sources away from open excavations and connected piping. Workmen should not be allowed to work in deep trenches unless the atmosphere is regularly checked for oxygen, methane, hydrogen sulfide, and other compounds of concern. Piping and other components should be checked for leaks before the system is put into service to prevent infiltration of air into the system. All construction and personal protective equipment must be decontaminated before use.

TABLE 6-2.

MATERIALS AND EQUIPMENT FOR ACTIVE LANDFILL GAS CONTROL SYSTEMS

Item	Materials	Installation
Well drilling	-----	Auger, caisson, or bucket drill rig
Well piping	2 to 6in. PVC, schedule 40 to 80, perforated and solid wall	Crane for deep wells, backhoe for shallow wells
Well backfill	1 in. washed crushed stone or river gravel	Place slowly by hand
Header piping	3 in. or greater (depending on flow/pressure requirements); PVC, polyethylene, or fiberglass (resistant to chemical attack)	Conventional trench excavating equipment, specialized jointing equipment for some pipe materials
Valves	Compatible with pipe size; gate, ball, or butterfly type; PVC or other reistant material	Note 1
Vacuum blower	Material or coating to resist chemical attack; size varies with flow/pressure requirements	Note 1
Safety devices	Specific items manufactured for use at refineries, sewage digestors, etc.	Note 1
Vent stacks	Any corrosion-resistant pipe of adequate size and strength, may require support	Note 1
Gas treatment	Note 2	Note 2

Notes:

1 - No special equipment required

2 - Addressed in Section 10.2

Source: SCS, 1980

6.2.5 Operation, Maintenance, and Monitoring

Active gas control systems require testing and adjustment throughout their lives of operation. Initial start-up testing is required to ensure that all components are functioning as intended. Throttling of individual well valves and blower control valves is required to "balance" the system. Mechanical components require regular service such as lubrication and part replacement. In addition, subsurface gas probes in the area being afforded protection should be monitored at least annually after system start-up to ensure that gas migration is being controlled (SCS, 1980).

Differential settlement of the landfill material beneath header pipes can cause pipe movements resulting in adverse slopes, accumulation of condensate in low spots, and partial or complete blockage of gas flow. Proper pipe slopes and condensate drains can minimize this problem. A regular program of periodic inspection and maintenance should be established to identify pipe breakage, condensate blockage, or other header system failure (Emcon, 1980).

Monitoring of the effectiveness of active gas control systems is conducted in the same manner as described in Section 6.1 for passive systems. Additional monitoring should be conducted during periods of system shutdown or operating problems. The interior of the blower/treatment facility should also be monitored, as leaks in the system can cause combustible gas to accumulate to dangerous concentrations.

6.2.6 Technology Selection/Evaluation

Active perimeter gas control systems are well-established as the most effective method of gas migration control, as subsurface gases are induced to flow to points of collection and cannot migrate beyond properly designed and operated systems. Because their performance is dependent on mechanical and electrical components, active systems can be less reliable, although possibly more effective, than passive systems. Blower capacity or extraction wells can readily be added to existing active systems to improve performance. Shutdown and other nonperformance alarms can be provided with active systems to identify the need for emergency maintenance and thus increase their reliability. All active systems require regular operation and maintenance associated with mechanical systems (motors, bearings, belts, etc.).

Active systems can be implemented with relatively conventional equipment, labor, and materials. Some mechanical equipment may require delivery periods of several months. Well drilling is affected with caisson, auger, and bucket rigs, and a few systems employing high torque capacity are needed to excavate through large obstacles that are present in landfills. Pipe-laying is similar to utility pipeline construction.

Worker safety is an important consideration where waste materials are being excavated. Personal protective gear must be used and provisions must be made for removing excavated waste materials and for decontaminating equipment. No flames or smoking should be allowed in the vicinity of open excavations.

Active gas control systems require relatively little time to implement. Several wells can be completed by a single crew in a day (equivalent to several hundred feet of perimeter per day), and collection piping and mechanical components can be installed concurrently. Gas control can be affected upon completion and start-up, and immediate results (as measured in monitoring probes) are realized.

6.2.7 Costs

The capital costs of active landfill gas control systems vary greatly, depending on the size and depth of the landfill, the nature of the waste, and the selected design criteria. Table 6-3 shows unit costs for typical active system components. The large range of unit costs is due to the variable nature of the system, depending upon the characteristics of the landfill in question. Unit costs for deep extraction wells will be greater than for shallower wells, due to the need for more specialized equipment. Likewise, large-diameter header pipe is more costly than smaller pipe due to higher material and labor costs. Blower/treatment facilities may vary in scale from a small blower with a vent stack to multiple, high volume blowers; multiple and/or high volume burners; and automatic timers, valves, switches, and recorders (SCS, 1980).

Annual operating and maintenance costs also vary with the size of the system. For example, a blower driven by a 5-horsepower motor operating continuously will consume about $2,000 to $3,000 worth of electricity at 5 cents/kilowatt-hour. Other electrical costs for lighting or automatic controls are nominal in comparison. The cost of replacement parts should also be small, since there are few mechanical components in the system. Small material costs can be expected for tools, lubrication, replacement of belts, fuses, etc. Manpower costs, assuming an average of two or three man-days per month on a contract basis, should be on the order of $5,000 annually; the costs will vary with the scale and sophistication of the system. Other annual costs that may be considered are insurance, security, interest, and administration or overhead (SCS, 1980).

Costs associated with treatment of collected gases will also contribute to the total cost of gas control. Treatment of gaseous waste streams is addressed in Section 10.2.

TABLE 6-3.
1985 UNIT COSTS FOR COMPONENTS OF ACTIVE LANDFILL GAS CONTROL AND COLLECTION SYSTEMS

Item	Unit	Cost Per Unit
Gas extraction well (drilling, stone, piping, etc.), in place	Vertical ft.	$50-75
Well connection lateral (10 ft. piping, valve, excavation fittings, etc.), in place	Each	$1,000-1,500
Gas collection header (piping excavation, fittings, etc.), in place	Linear ft.	$20-100
Blower facility (blower(s), safety devices, valves, foundation, piping, fencing electrical components, and service connection), in place	Lump sum	$50,000-100,000
Monitoring probe (drilling, pipe, fittings, backfill, etc.), in place	Vertical ft.	$10-15
Operation and maintenance	Year	$5,000-20,000
Monitoring with portable meter	Each visit	$10-15

Source: Flood, F. SCS Engineers, Reston, VA, personal communication, March 1985.

6.3 Active Interior Gas Collection/Recovery Systems

6.3.1 General Description

Gases generated at landfill sites and vented to the atmosphere frequently contain malodorous and/or toxic compounds. Malodors are most commonly associated with trace concentrations of natural organic and sulfurous gases. Toxic compounds may be present as a result of volatilization of man-made organic compounds that are part of the landfilled waste material.

The following is an example of an estimate of the costs of any active perimeter gas migration control system:

Scenario:

- Closed landfill containing primarily domestic refuse; free liquid organic wastes are mixed with solid waste.
- Average landfill depth is 40 feet.
- Landfill perimeter is 2,300 feet.
- Continuous low-permeability clay exists at depths of 25 feet and underlies the bottom of the landfill.

Required:

Estimate the capital and O&M costs of an active perimeter gas migration control system.

Assumptions:

- Extraction wells will be installed in landfilled material and will be 30 feet deep.
- The line of wells will be inset from the edge of the landfill and will be 2,100 feet long (parallel with and inside of the landfill edge).
- Wells will be spaced at 50 feet.
- Monitoring probes will be spaced at 100 feet, will be 30 feet deep, and will be along a line 2,500 feet long (parallel with and inside of the landfill edge).
- Probes will be monitored quarterly with a portable meter.

Estimates of Quantities:

- Gas extraction well: 2,100' ÷ 100' x 30' = 630 vertical feet

- Well connection lateral: 2,100' ÷ 100' = 21 laterals
- Gas collection header: 2,100 linear feet
- Blower facility: 1 facility
- Monitoring probes: 2,500' ÷ 100' x 30' = 750 vertical feet
- Monitoring: 2,500' ÷ 100' x 4 visits/year = 100 visits/year

Estimate of Capital Costs:

Item	Quantity	Unit Cost Low	Unit Cost High	Item Cost Low	Item Cost High
Gas extraction well	630 v.f.	$ 50	$ 75	$31,500	$ 47,250
Well connection lateral	21 laterals	$ 1,000	$ 1,000	$21,000	$ 31,500
Gas collection header	2,100 l.f.	$ 40 avg.*	$ 60 avg.*	$ 84,000	$126,000
Blower facility	1 facility	$50,000	$75,000	$ 50,000	$ 75,000
Monitoring probes	750 v.f.	$ 10	$ 15	$ 7,500	11,250
Total Capital Costs				$194,000	$291,000

Estimate of Annual O&M Costs:

Item	Quantity	Unit Cost Low	Unit Cost High	Item Cost Low	Item Cost High
Operation and maintenance	1 lump sum	$ 5,000**	$10,000**	$ 5,000	$ 10,000
Monitoring with ·portable meter	100 visits	$ 10	$ 15	$ 1,000	$ 1,500
Total annual O&M Costs				$ 6,000	$ 11,500

**Control system is comparatively small; O&M costs on low end of range in Table 6-3.

Gases generated in landfills will ultimately vent to the atmosphere, either vertically through the cover material, laterally through surrounding soil outside of the landfill limit and then vertically through paths of low resistance to flow, or through perimeter (active or passive) control systems. Gases that vent from landfills are generally not detected and presumably do not significantly degrade local air quality because of dilution in the atmosphere. However, emissions from some landfills do measurably degrade air quality, and experience with landfill-generated methane recovery and utilization systems has shown that landfill gas collection generally affords a net improvement in local air quality. Installation of several landfill gas collection systems has been justified, at least in part, by a recognized need to reduce emissions from the landfills (Johns Hopkins University, 1980).

Where the venting of landfill gas poses a health or environmental problem, systems designed specifically for the collection of gases beneath the landfill surface (before venting occurs) can be installed. Such systems are similar to perimeter active control systems in their basic concept of operation; however, gas extraction wells and corresponding collection headers are placed over the entire landfill surface as shown in Figure 6-8. Interior gas collection/recovery systems are intended to capture as much as possible to prevent hazardous gases from being emitted to the atmosphere. Collected gases are conveyed to a central point for treatment or destruction (see Section 10.2) or for processing and recovery. Over 50 systems that recover landfilled-generated methane for a variety of beneficial uses are in operation or are under development in the United States, Europe, and Canada (Waste Age, 1984). Emcon (1980, 1981) provides additional information specific to recovery and utilization of landfill-generated methane gas.

6.3.2 Applications/Limitations

Active interior gas collection/recovery systems are applicable to landfill sites where gaseous emissions through the surface are to be controlled. This method can be used to supplement landfill capping (see Section 3.1) and to prevent resulting lateral gas migration. They can be used at virtually any site where it is possible to drill or excavate through landfilled material to the required depth. Limiting factors could include the presence of free-standing leachate (i.e., saturation) or impenetrable materials within the landfill.

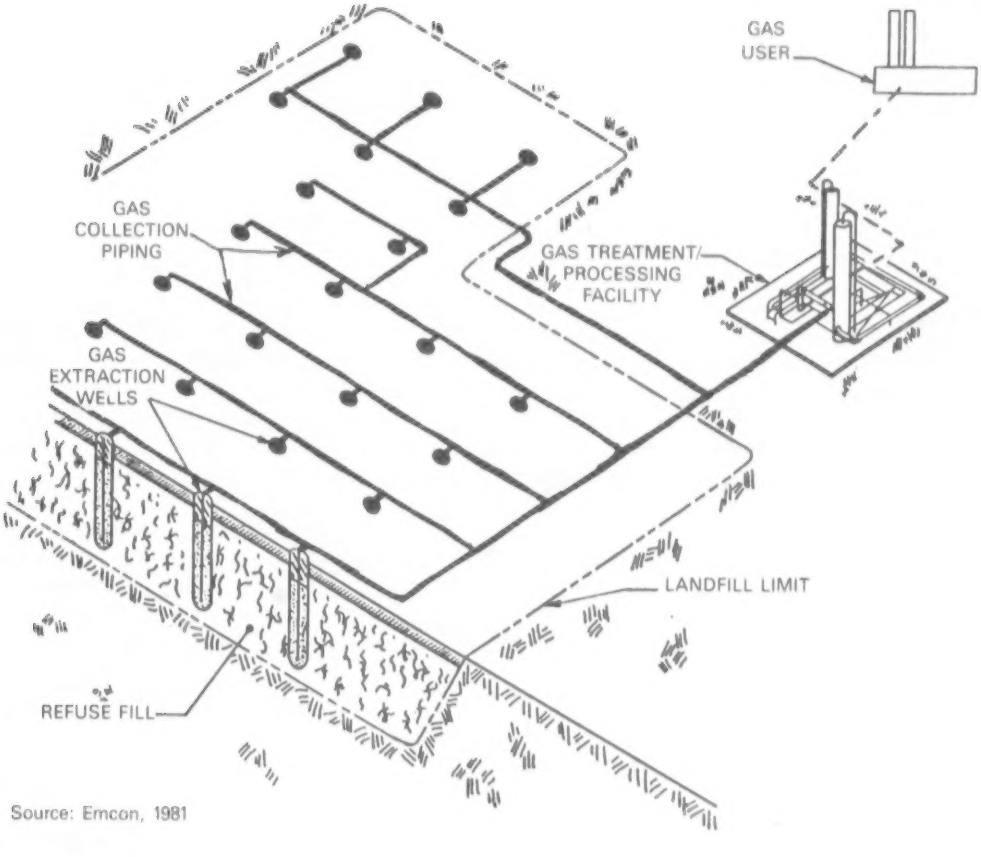
6.3.3 Design Considerations

The design of landfill gas collection systems requires consideration of the same basic criteria as described under active gas control systems (see Section 6.2). Major variations are described herein.

The depths of gas extraction wells used for gas collection generally do not need to extend to the bottom of the landfill; typical depths range from 50 to 90 percent of the landfill depth at the well location. In addition, the presence of free-standing liquid waste or leachate within the landfill can limit the practical depth of well installation.

Well spacing is determined in a manner similar to that described in Section 6.2. Spacing of extraction wells is generally greater because greater interior depths allow higher applied vacuum levels and larger "radii of influence". Well spacings of 200 feet are common for gas collection. The well system is laid out to cover the landfill surface in configurations that allow complete coverage with the minimum number of wells, ideally in an

FIGURE 6-8.
GAS COLLECTION/RECOVERY SYSTEM



Source: Emcon, 1981

equilateral triangular pattern. The geometry of the landfill surface generally precludes the ideal well layout, requiring compromises and approximations (Emcon, 1980).

The design of the gas collection header system is more complex for collection systems because of the network of branches required to reach all extraction wells. However, the same basic procedure of flow/pressure calculations is followed.

6.3.4 Construction/Implementation Considerations

The considerations discussed in Section 6.2 for the construction and implementation of active perimeter gas control systems installed within landfilled material also apply to active interior gas collection/recovery systems.

6.3.5 Operation, Maintenance, and Monitoring

Active interior gas collection/recovery systems require essentially the same operation and maintenance as active perimeter gas control systems. The major difference is a matter of scale; collection/recovery systems generally have more wells, piping, associated gas flow, and condensate water. The relative complexity of the piping network requires frequent measurements to balance gas flow and pressure among the extraction wells.

Monitoring the effectiveness of interior gas collection/recovery systems differs substantially from monitoring perimeter systems because the problems relieved by the systems differ. Measurement of air contaminants, upwind, directly above, and downwind of the site is required to determine the effectiveness of such systems. Air quality monitoring is not addressed herein.

6.3.6 Technology Selection/Evaluation

Active interior gas collection captures gases before they can vent from a landfill and thus is the only method of limiting gaseous emissions from landfills. The technology has been shown to improve local air quality (where the gas is subsequently treated or recovered), regardless of whether air quality was the primary concern. Because the method relies on mechanical components, however, shutdown alarms and regular maintenance and monitoring should be provided to affect maximum system reliability. Blower capacity or extraction wells can readily be added to existing active systems to improve performance. All active systems require regular operation and maintenance associated with mechanical systems (motors, bearings, belts, etc.).

Active systems can be implemented with relatively conventional equipment, labor, and materials. Some mechanical equipment may require delivery periods of several months. Well drilling is accomplished with caisson, auger, and bucket rigs, and a few systems employing high torque capacity are needed to excavate through large obstacles present in landfills. Pipe laying is similar to utility pipeline construction.

Worker safety is an important consideration where waste materials are being excavated. Personal protective gear must be used and provisions must be made for removing excavated waste materials and for decontaminating equipment. No flames or smoking should be allowed in the vicinity of open excavations.

6.3.7 Costs

Unit costs of active interior landfill gas collection systems are comparable to those of active control systems (see Table 6-3). Overall costs are generally higher for collection systems because of the larger number and depth of wells and larger diameter and length of header pipe needed to collect gas from the entire landfill, rather than only along the perimeter.

Costs associated with treatment of collected gases will also contribute to the total cost of gas control. Treatment of gaseous waste streams is addressed in Section 10.2.

Costs of active interior landfill gas collection systems are estimated in a manner similar to costs of active perimeter gas migration control systems. For the example scenario presented in Section 6.2.7, the number of wells would be estimated in accordance with the assumed spacing; (e.g., 200-foot triangular pattern, one well per acre, etc.), and the length of header pipe would be estimated accordingly. A larger and more costly blower facility would be required to handle higher rates of gas flow. Also, if gas migration control were not part of the performance requirement, monitoring probes would not be installed.

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SECTION 7

ON-SITE AND OFF-SITE DISPOSAL OF WASTES AND SOIL

This section discusses excavation and removal and on-site and off-site land disposal of hazardous wastes and contaminated soil. Excavation, removal, and hauling of wastes and soil at hazardous waste sites are generally accomplished with conventional heavy construction equipment. This equipment is also commonly used for construction of land disposal facilities. Section 7.1 discusses the equipment used for excavating, lifting, loading, hauling, dumping, and grading on-site soil and waste material. Section 7.2 provides a brief overview of off-site waste disposal. Section 7.3 gives general guidance for the design and construction of on-site disposal and storage facilities to comply with RCRA standards.

7.1 Excavation and Removal

This section describes conventional equipment and the methods applicable for excavation and removal of contaminated soils, sludges, and liquids.

7.1.1 General Description

As noted in Section 7.1.3, the major types of excavation and removal are casting and loading excavation and hauling excavation (including the use of pumping systems). The equipment described in this section may be used for constructing an on-site disposal facility as well as for excavation and removal of contaminated materials. A more detailed description of excavation equipment can be found in Church (1981).

7.1.2 Applications/Limitations

Excavation and removal followed by land disposal or treatment are performed extensively in hazardous waste site remediation. There are no absolute limitations on the types of waste which can be excavated and removed. However, worker health and safety weighs heavily in the decision to excavate explosive, reactive, or highly toxic waste material. Other factors which are considered include the mobility of the wastes, the feasibility of on-site containment or in-situ treatment, and the cost of disposing the waste or rendering it non-hazardous once it has been excavated. A frequent practice at hazardous waste sites is to excavate and remove contaminant "hot spots" and to use other remedial measures for less contaminated soils.

Excavation and removal is applicable to almost all site conditions, although it may become cost-prohibitive at great depths or in complex hydrogeologic environments.

Each of the equipment types discussed in Section 7.1.3 have specific applications and limitations which must be carefully evaluated.

7.1.3 Excavation Equipment

7.1.3.1 Loading and Casting Excavation

Loading and casting can be accomplished by a wide variety of conventional equipment ranging in size from a 220 cubic yard dragline down to the 1/4 cubic yard backhoe (Church, 1981). These basic types of excavation machinery fall into the following general categories:

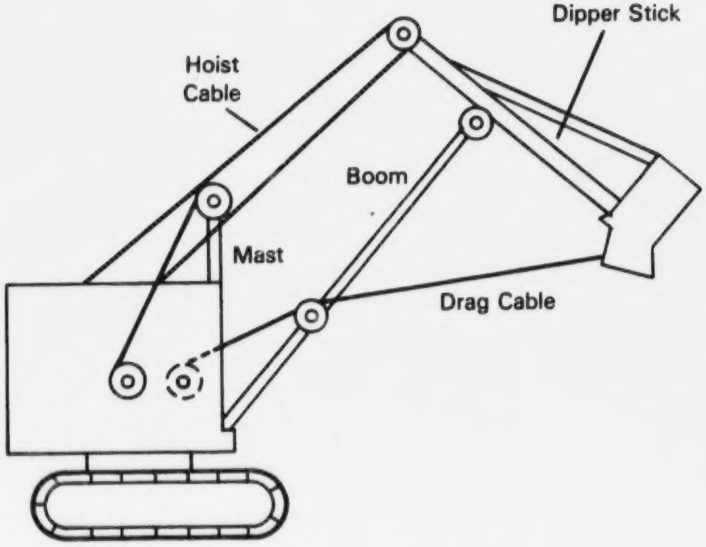
- Backhoes
- Cranes and attachments (draglines and clamshells)
- Dozers and loaders.

It should be noted that these are not the only excavators used for loading and casting. Other types of loading and casting equipment include trenchers, belt loaders, and wheel bucket excavators (Church, 1981). This equipment is not discussed in this section because it has very limited use in excavation at a hazardous waste site.

a. Backhoes

The backhoe unit is a boom or dipper stick with a hoe dipper attached to the outer end. Figure 7-1 illustrates the components of the backhoe. The unit is generally a crawler-mounted, hydraulically operated vehicle with various sized-toothed buckets ("dippers") attached to the boom, and dipper-arm assemblies of varying lengths. Backhoes are generally used for trenching and subsurface excavation where it is expedient to keep the excavator at the original ground level (Church, 1981). However, where it is necessary for the backhoe to excavate beyond the maximum depth of the boom and dipper assembly, a "working bench" can be excavated for the backhoe next to the trench so that the vehicle can excavate to the desired depth. The backhoe unit can also be adapted with various attachments such as grapples for drum excavation work.

FIGURE 7-1. TYPICAL BACKHOLE



Source: EPA, 1978

The digging dimensions of a backhoe are shown in Figure 7-2. The maximum reach and depth for various sized hoes is shown in Table 7-1. As shown in this figure, the largest backhoe will dig to a maximum depth of about 30 feet. Deeper digging depths (up to 80 feet) can also be achieved by using backhoes with extended dipper sticks, modified engines, and counterweight frames.

Smaller backhoes with rubber tires are useful for fast excavation on stable working surfaces. One frequently used smaller backhoe is a wheel-mounted combination backhoe and front-end loader. This vehicle can excavate, lift, load, haul, and dump soil and waste materials (including both crushed and undamaged drums). Its operation, however, is generally restricted to relatively flat and stable working surfaces.

Theoretical Production Rate--The hourly production in cubic yards bank measurement (cy bm) per 50 minute working hour for a backhoe and dragline can be expressed by the equation (Church, 1981):

$$P = \frac{50}{CT} \times BF \times BC$$

where: P = hourly production (cy bm)
CT = cycle time (min)
BF = bucket factor
BC = bucket capacity.

The cycle time varies according to length of hoist and angle of swing return. For a backhoe having a hoist length of 10 feet and swing angles of 30 and 180 degrees, the cycle times are 0.37 and 0.52 minutes, respectively. Similarly, a backhoe with a hoist of 60 feet and swing angles of 30 and 180 degrees has cycle times of 1.20 and 1.35 minutes respectively (Church, 1981).

TABLE 7-1.
MAXIMUM REACH AND DEPTH FOR VARIOUS SIZED HOES
(MAXIMUM DIGGING ANGLE OF 45°)

Hoe size (yd^3)	Maximum reach of boom (ft)	Maximum depth of excavation (ft)
1	35	22
1-1/2	42	25
2	49	30
3-1/2	70	45

Source: USEPA, 1978

FIGURE 7-2.
HOE DIGGING RANGES

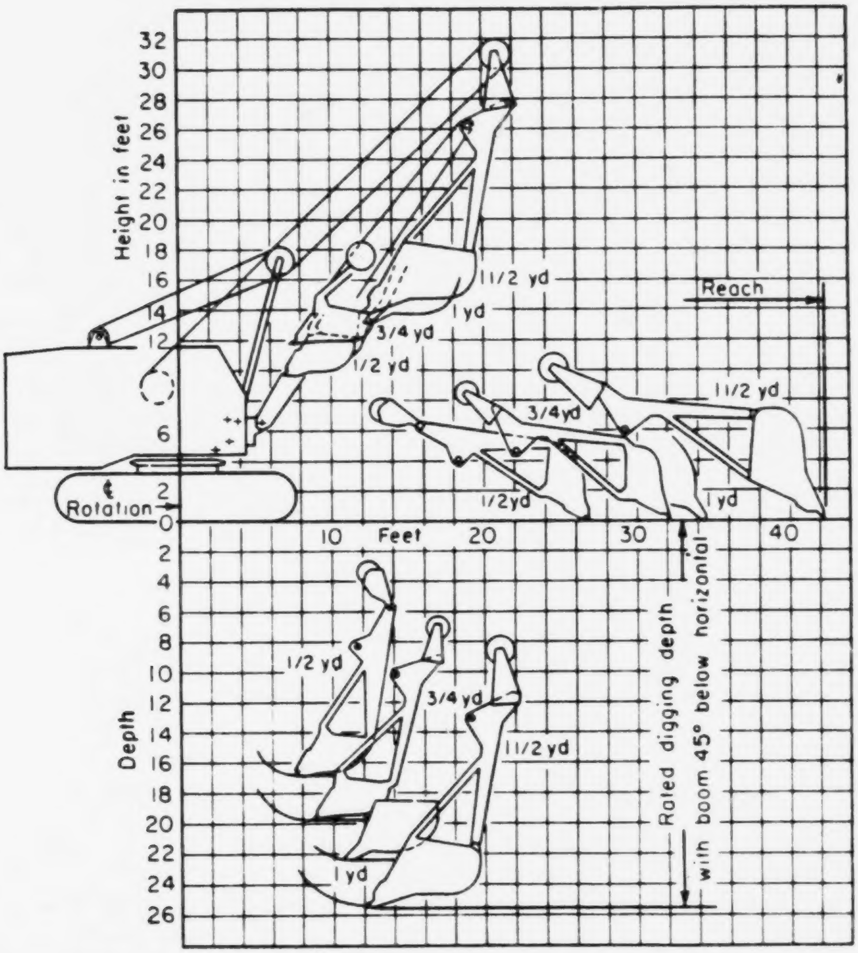


FIGURE 7-1 Source: Stubbs, 1959

The bucket factor varies according to the nature of rock-earth. For calculation purposes, an average bucket factor of 0.66 will be assumed (Church, 1981). An estimate for the hourly production of a backhoe having a 1-1/2 yd^3 bucket and a 40 foot hoist with a 180 degree swing angle can thus be shown to be:

$$P = \frac{50 \text{ cy bm}}{1.02 \text{ min}} \times 0.66 \times 1.5 \text{ yd}^3$$

$$= 49 \text{ cy bm/hr.}$$

Costs--The total 1985 unit cost for a backhoe including labor, equipment, and overhead and profit varies from $1.64 to $2.44 for bucket capacities of 3.5 and 1 yd^3, respectively (Godfrey, 1984). The daily output index is 1200 yd^3 and 360 yd^3, respectively. This means that the total daily cost for backhoe utilizing a 3.5 yd^3 bucket is approximately $1970.

b. Cranes and Attachments

The crane equipped with a clamshell or orange-peel bucket is rarely used for loading or casting excavation in the sense of high production. Its uses are in subaqueous excavation and in the rehandling of materials. For instance, cable-operated cranes fitted with the clamshell buckets, drum grapples, magnets, hoists, slings, and lifters are ideal for large-scale drum excavation, lifting, and staging at sites with unrestricted working space.

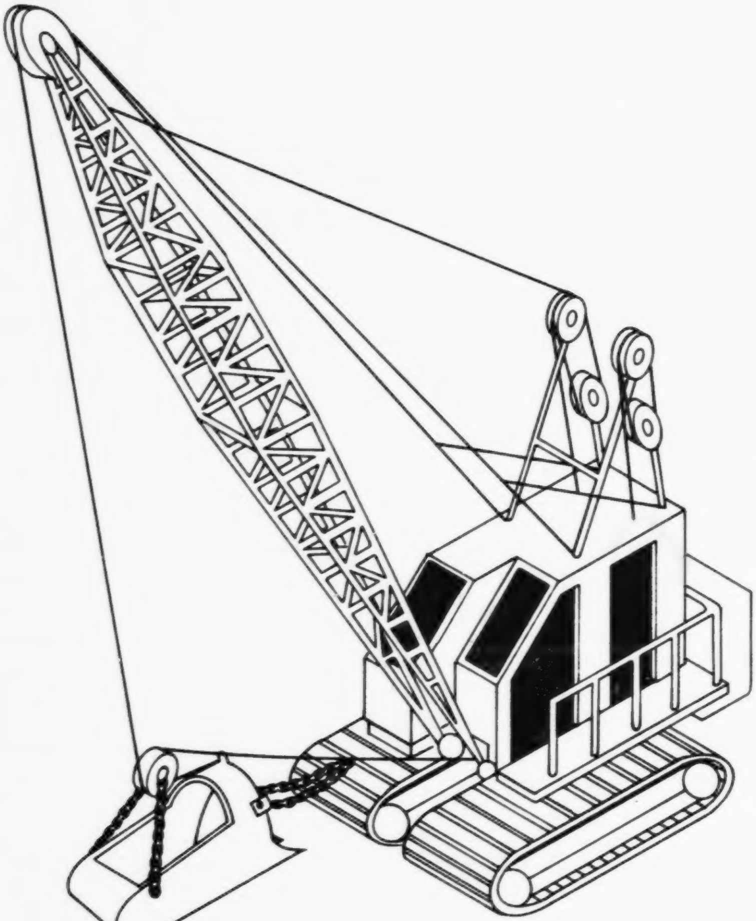
Cranes can also be adapted for use as dragline excavators for deeper excavations over large areas. A dragline excavator is a crane unit with a drag bucket connected by cable to the boom. A dragline is illustrated in Figure 7-3. The bucket is filled by scraping it along the top layer of soil toward the machine by a drag cable. The dragline can operate below and beyond the end of the boom.

Maximum digging depth of a dragline is approximately equal to half the length of the boom, while digging reach is slightly greater than the length of the boom (USEPA, 1978). Drag buckets can vary in size from 1 yd^3 to 20 yd^3, with boom lengths ranging from 30 to 240 feet (Church, 1981). Various working dimensions of draglines for various bucket sizes are shown in Table 7-2.

Draglines are very suitable for excavating large land areas with loosely compacted soil. Excavation with draglines of landfill sites containing explosive materials or very toxic chemicals is unsafe.

Theoretical Production Rate--The hourly production rate for a dragline excavator is calculated in the same way as for the backhoe. The production rate based on a 50 minute working hour can range from 160 cy bm (for bucket capacity of 1 yd^3, swing-return angle of 30 degrees, and a hoist of 10 feet) to 510 cy bm (for hoist length of 200 ft, swing-return angle of 180 degrees

FIGURE 7-3.
A DRAGLINE



Source: EPA, 1978

TABLE 7-2

TYPICAL DRAGLINE EXCAVATOR DIMENSION

Item	Bucket size in cubic yards (CY) 3/4	1	1-1/4	1-3/4	2
Dumping radius, ft	30	35	36	45	53
Dumping height, ft	17	17	17	25	28
Maximum digging depth, ft	12	16	19	24	30
Digging reach, ft	40	45	46	57	68
Boom length, ft	35	40	40	50	60
Bucket length, ft	11.5	14.67	11.83	13.08	14

Note that these values apply to operation of the excavator with its boom at a 40° angle to the horizon.

Source: EPA, 1978

and a bucket capacity of 20 yd^3). The hourly production rates obtained assume that the ground is average weathered rock-earth (Church, 1981).

Costs--The total 1985 cost for labor and equipment (including overhead and profit) for a dragline with a 3/4 yd^3 bucket capacity was $3.00/$yd^3$ at a production rate of 35 cubic yards per hour. Cost for a dragline with a bucket capacity of 3 yd^3 with a production rate of 112 cubic yards per hour is $1.46/$yd^3$ (Godfrey, 1984). The output index for the 3 yd^3 bucket is 900 yd^3/day, leading to a total cost of $1300/day.

c. Dozers and Loaders

Dozers and loaders are generally equipped with a hydraulically controlled (versus mechanical cable hoist) blade and bucket lift and can be either crawler- or rubber-tire-mounted. Crawler machines are equipped with self-laying steel tracks of variable cleat design and width, which provide good ground contact and excellent flotation and traction capabilities. For this reason, crawlers are ideally suited for excavating over rough, unstable surfaces. In marshy or swampy areas where mobility is limited, extra wide tracks are recommended to improve traction.

Dozers and loaders are also available with large rubber-tired wheels that are faster and more mobile than crawler machines on level terrain. Their ability to maneuver on rough, muddy, and sloping terrain, however, depends somewhat on the type of tires. For example, tires with a wide base and low air pressure provide good flotation and traction (Church, 1981).

Crawler dozers equipped with blades of various sizes and shapes (straight to U-shaped) have tremendous earth-moving power and are excellent graders. In drum excavation work, these dozers can remove miscellaneous fill or soil overburden, or they can push earth and undamaged or empty drums from unstable surface areas to more accessible areas for lifting and loading operations. The dozers are usually used in combination with other excavation equipment such as backhoes.

Front-end loaders are tractors equipped with buckets for digging, lifting, hauling, and dumping materials. Both crawler-mounted and rubber-tired front-end loaders are widely used in hauling and staging undamaged drums. Because lifting and loading drums onto front-end loaders usually requires manual assistance, their use should be limited to structurally sound drums.

The crawler loader is an excellent excavator that can carry materials as far as 300 feet (Brunner and Keller, 1972). Front-end buckets vary in capacity and design. Medium-sized crawler loaders typically have maximum bucket capacities of 5 to 6 cubic yards. Wheel-mounted bucket loaders, for high-production operations on stable surfaces such as paved areas, have bucket capacities up to 20 cubic yards.

Theoretical Production Rate--The hourly production rate can be calculated in the same manner as the backhoe and the dragline. A medium sized crawler loader having a bucket capacity of 5 cubic yards, with an average cycle time of 50 minutes for a working hour, and a bucket factor of 0.66, would have a production rate of 330 cubic yards bank measurement per hour.

Cost--A wheel mounted bucket loader having a bucket capacity of 5 cubic yards has a total 1985 unit cost of \$0.84/yd^3 (Godfrey, 1984). The daily output would be 1,480 cubic yards, and, therefore, the total costs for labor, equipment, and overhead and profit would be \$1240/day.

7.1.3.2 Hauling Excavation

Hauling excavation is used for on-site and off-site transport of wastes. The hauling equipment discussed in this subsection includes scrapers and haulers. Dredges are also used for hauling excavation and are discussed in Section 8. Dozers and loaders discussed in the previous section can also be used for hauling.

a. Scrapers

Wheel-mounted scrapers are generally used in excavation work to remove and haul surface cover material at large disposal sites. However, they are not suitable where drums are buried near the surface. They are also useful in respreading and compacting cover soils.

Scrapers are available as both self-propelled, self-loading vehicles, and as models that are push-loaded by crawler tractors. Soft- to medium-density cover soils and fill favor the self-loading scraper; medium to hard rock and earth favor the use of the push-loaded machine. The hauling capacities of scrapers range from 2 to 40 cubic yards. These earth moving machines can haul cover material economically over relatively long distances--more than 1,000 feet for self-propelled scrapers (Church, 1981).

Costs--Total 1985 unit costs including labor, equipment, overhead and profit vary from $2.11/cubic yard to $3.98/cubic yard for self-propelled scrapers (15 yd^3 capacity, 1/4 dozer, 1500 foot haul) and towed scrapers (10 yd^3 capacity, 1/4 push dozer, 5000 foot haul), respectively (Godfrey, 1984). The corresponding daily output is 800 yd^3 and 440 yd^3, respectively, leading to total daily costs of $1670/day and $1750/day.

b. Haulers

A variety of haul trucks are available for transporting excavated materials and waste drums, both off-the-road and on-the-road. Haulers are large, rubber-tired vehicles available as single-trailer, 2- or 3-axle vehicles, and as double-trailer, multiple-axle haulers. Their rated haul capacities range from 1 to 100 tons, and they are available as bottom-dump, rear-dump, and side-dump vehicles. Small, 1 to 2 ton haul trucks are used most commonly in drum transport operations.

At hazardous waste disposal sites, haul trucks are most useful for hauling excavated soils and drums (damaged or undamaged) to off-site secure landfills or selected drum reburial sites. Soil can be loaded onto haulers using backhoes, draglines, shovels, and loaders. Drums can be loaded onto and removed from haulers using backhoes, cranes, and forklift trucks, usually with manual assistance from field workers. Barrel grapplers, however, can usually perform this task without manual assistance.

Costs--Total 1985 unit costs for dump trucks vary according to capacity of truck and the distance of hauling, which decreases the daily output. For instance, the costs for a 6 yd^3 dump truck, hauling 1/4 mile round trip, has a daily output of 240 yd^3 and a total unit cost of $1.78/yd^3. The same truck hauling 4 mile round trip has a daily output of 85 yd^3 and a total unit cost of $5.05/yd^3. A 12 yd^3 dump truck hauling 1/4 mile has a daily output of 356 yd^3 and a total unit cost of $1.38/yd^3. The same truck hauling 20 mile round trip has a daily output of 32 yd^3 and a total unit cost of $15.40 yd^3 (Godfrey, 1984).

7.1.3.3 Pumps

a. General Description

Pumping is required in order to remove liquids and sludges from ponds, waste lagoons, and surface impoundments. Liquid wastes pumped from these sites must be managed so as to prevent degradation of the surrounding environment. The liquid wastes may be pumped to a treatment system or a tank truck for transport off-site to a commercially operated treatment facility. The following section discusses types of pumps available, factors affecting selection, and costs of temporary pumping systems to be used for remedial action.

Pump Types--Pump types may be divided into two major categories: (1) dynamic, in which energy is continuously added to increase the fluid velocity; and (2) displacement, in which energy is added periodically (Karassik et al., 1976). Dynamic pumps may be further subdivided into several varieties of centrifuge pumps while displacement pumps can be subdivided into reciprocating and displacement or rotary pumps.

Centrifugal Pumps--Dynamic centrifugal pumps have a wide range of capacities; flow rates range from 2 or 3 gpm up to 10,000 gpm (Perry and Chilton, 1973). The primary advantages are low initial cost and a simple design, which in turn means low maintenance and easy repair. These pumps are best suited for pumping large volumes against small heads (Cole-Parmer, 1982). They can handle liquid with large amounts of solids. Centrifugal pumps are limited in that they are not self-priming, and therefore liquid must be added to the pump in order to start up the pumping action (i.e., they cannot pump dry).

Some pumps are coated with special engineering plastics such as PVC, PVDF (Kyner) or polypropylene to handle corrosives, caustics, dyes, brines, halogenated materials slurries, and other wet substances. These plastics are compatible with most chemicals.

Criteria for selecting centrifugal pumps include waste composition, chemical compatabilities, service and operating conditions such as flow rate and discharge head, temperature range, and power requirements, and nature of the solids such as abrasiveness and viscosity.

Reciprocating Pumps--Diaphram, bellow and piston pumps are subcategories of reciprocating pumps. Reciprocating pumps displace a precise volume of liquid with each movement of an inlet suction and outlet discharge cycle. Reciprocating pumps have the advantage of being able to deliver fluids against high pressures and operate with high efficiencies over a wide range of operating conditions. The capability of achieving high pressures at low velocities is important when pumping abrasive slurries or other high viscosity fluids.

Diaphram pumps pulse a flexible membrane to displace the liquid with each stroke. The pump is usually driven by an outside source which may be at a constant or variable speed and it may be driven mechanically or hydraulically. These pumps can be used for product pressures up to 150 psi.

Applications of diaphram pumps are more numerous than centrifugal pumps. Because they have no seals, they can handle fluid mixtures with a higher percentage of solids, such as mud, silt and sludge. The leakfree nature of this pump frequently causes it to be selected to prevent cross-contamination when the fluid to be pumped is abrasive or corrosive (Perry and Chilton, 1973, Karassik, 1976).

Bellow pumps operate by moving a bellow back and forth to displace the liquid. These pumps can be used to produce pressures of up to 50 psi. Applications of bellow pumps are more restricted than diaphram pumps, but because these pumps have no seals and special nonclogging valves are available for them, abrasive or particulate mixtures can be pumped (Perry and Chilton, 1973).

Piston pumps use a reciprocating plunger to draw in and force out fluids. These pumps are used where pressures of 600 to 10,000 psi are needed.

Piston pumps are not recommended for use with abrasive fluids because these pumps require a packing seal to prevent leaks. Since in some piston pumps, the piston and cylinder are open to the fluid being pumped, they are not recommended for use with corrosive chemicals. Piston pumps have the desirable characteristic of maintaining high volumetric efficiency at any desired flow rate (Karassik et al., 1976). Volumetric efficiency is the ratio of liquid actually pumped to that which theoretically should be moved, based on piston displacement (Perry and Chilton, 1973).

Reciprocating pumps have the advantages of being able to pump sludges. Additionally, total costs including initial, power, and maintenance are lower than comparable pumps. Another advantage is that when compared to centrifugal and displacement pumps, the reciprocating pumps have been determined to be the least sensitive to changes in capacity when the discharge pressure varies. One reason for this is because leakage past the plunger seals and check valves is comparatively small. Another reason to use reciprocating pumps is the ease with which the capacity can be accurately adjusted with the aid of a metering device (Henshaw, 1981).

Disadvantages of reciprocating pumps include pulsating flow. This may cause a problem when the fluid to be pumped needs to be steadily entrained. Pulse dampeners are on the market to reduce this pulsation by 90 percent. For most applications initial and maintenance costs will be greater than centrifugal pumps, but total costs will be less, as described previously. Most problems with reciprocating pumps can be avoided by selecting pumps appropriate for the particular job (Perry and Chilton, 1973; Karassik, 1976).

Selection considerations for choosing a reciprocating pump are: intake and discharge pressures, metering capabilities, properties of the liquid to be pumped (abrasive, corrosive, pH, etc.), temperature, type of flow, power, maintenance, amount of cleaning needed, and cost.

Displacement Pumps--The third type of pumps are the positive displacement pumps, which consist of gear, flexible impeller and flying vane pumps. These types of pumps move a fixed volume of liquid from the inlet to the outlet by the pass of gear teeth or impeller blades.

Gear pumps consist of two meshed spur gears in a circular housing. As gears mesh and rotate, the fluid is forced out of the spaces between the teeth. Gear pumps produce a virtually pulseless, low shear flow, even against moderately high heads; and they require no check valves. These pump can be used to produce pressures up to 100 psi.

Because of the close running of gears, gear pumps are not suitable for abrasive substances. Applications do include a wide range of fluids, including some corrosive chemicals.

Flexible impeller pumps are versatile and efficient. A flexible, vaned membrane, usually made of rubber, rotates in an eccentric housing to draw the liquid in and force it out. The volumes of the spaces change as the impeller rotates. Different grades of impellers are compatible with fluids with pressures up to 30 psi. Flexible impeller pumps cannot handle abrasives but will pump various liquids. Flexible impeller pumps combine general features of gear and centrifugal pumps. They are almost as efficient, require no check valves and are self priming.

Flying vane pumps use movable vanes in place of flexible impellers. Flying vane pumps also combine the general features of gear and centrifugal pumps. They are almost as efficient, are self priming and require no check valves. These pumps have low maintenance costs.

Positive displacement pumps are designed with close tolerances between the pump house and the impellers so the liquid being pumped cannot leak back around the impeller. For this reason, these pumps can develop pressure differences of up to 100 psi in gear pumps and slightly less for other displacment pumps. This close tolerance utilizes the liquid being pumped to self prime the pump and lubricate the impellers, so this type of pump should not be run dry.

Applications of displacement pumps are varied. Generally, these pumps are not to be used with abrasives.

Selection considerations for choosing a displacement pump are: required flow, intake pressure, exit pressure, properties of the liquid, temperature, power intake, maintenance requirements, cost, and ease of cleaning.

Immersion Pumps--Immersion pumps are designed so that the inlet port is immersed in liquid but the motor and electrical components remain dry. This type of pump is virtually maintenance free and for the most part no metal contacts the pumped liquid so corrosives can be pumped. Parts of this pump can be constructed of materials suitable to the liquid being pumped so that they are applicable for use with hard to handle chemicals such as sulfuric acid, sodium hydroxide and ferric chloride. These pumps are self priming and can pump liquids of temperatures up to 260°.

Submersible Pumps--Submersible pumps operate only when totally submersed in the fluid which is being pumped. They contain liquid-proof electrical connections and use a motor which is cooled by the liquid. These pumps are economical and energy efficient. Applications include industrial process wastewater, flood water and most clean or dirty waters. Some submersible pumps are built to pump mildly corrosive solutions and kerosene based solutions. Certain types of submersible pumps can work in as little as 3/16 inches of liquid and some can pump semi-solids of appreciable size. Costs for submersible pumps are moderate and depend on pumping needs.

Selection considerations for submersible pumps include required flow, properties of the liquid, power capabilities, temperature, ease of cleaning and maintenance, and costs.

b. Pump Selection

In selecting a pump, care must be taken to ensure reliable operation and control. Pump selection generally depends on the pumping capacity required and the materials to be pumped. The experience of pump manufacturers is often valuable in selecting the proper size and type of pump and motor.

When dealing with hazardous wastes, the task of pump selection is complicated by the presence of chemicals that could corrode or dissolve pump parts. Corrosive liquids having a low pH or a high chloride ion content can rapidly destroy most metal pumps. Wetted parts should be plastic, rubber, or ceramic, or, if made of iron, should be alloyed with silicon and/or chromium (Beck, 1984). It is extremely important to check the chemical compatibiliy of seals with the fluid being pumped.

The presence of abrasive liquids also influences pump selection. Internal passages must have adequate dimensions or abrasive particles will damage parts that they rub against. Close internal clearances between stationary and moving parts is undesirable. Rubber and ceramic parts resist abrasive wear better than metal parts (Perry and Chilton, 1973). Many manufacturers make abrasion-resistant models, and the pump should be selected after a detailed assessment of the waste to be pumped has been made.

The pump is not the only critical component in a properly designed pumping system. Other factors, such as pipe sizes and configurations, fittings, type of hose used, and nature of the substance being pumped can determine to a large degree the efficiency of a pumping system.

c. Costs

This section presents 1985 cost data for rental of a centrifugal pumping system. These data are best estimates and individual vendors should be contacted for specific costs. Operating costs are determined by the size of the pump (i.e., horsepower) and the number of hours of operation.

Power costs for a pump are determined using the following equations:

$$(1)\quad \text{Power consumption} = \frac{\text{bhp}}{E_m}$$

bhp = brake horsepower
E_m = efficiency

$$(2)\quad \text{Power cost} = \frac{\text{bhp}}{E_m}\ (0.7457\ \text{kw/hp})\ (\text{cost/kWh})\ (\text{operating hours})$$

kWh = kilowatt-hour
hp = horsepower

Power costs for 10, 50, and 100 bhp pumps are \$932, \$4600, and \$9320, respectively. These costs are based on an 80 percent operating efficiency, 1000 hours of operation, and an electricity cost of \$0.10/kilowatt-hour (Peters and Timmerhaus, 1980). Operating costs are determined by the size of the pump (i.e., horsepower) and the number of hours of operation. Hourly operation costs are estimated to range from \$0.42/hour to \$1.63/hr for centrifugal pumps having capacities of 400 gpm to 90,000 gmp (Godfrey, 1984). The total rental costs vary according to daily, weekly, and monthly rates. The costs vary from \$16/day to \$135/month for a pump having 400 gpm capacity (Godfrey, 1984). It should be noted that these are approximate costs, and final costing would require more detailed analysis.

7.1.3.4. Industrial Vacuum Loaders

Industrial vacuum loaders such as the "Supersucker" (Super Products, undated) and the "Vactor" (Peabody-Myers, undated) can be used in large-scale cleanup operations to remove soil or pools of liquid waste. Using industrial loaders for soil removal is safer and more efficient than using hand tools. The Supersucker and the Vactor are vehicle-mounted, high-strength vacuums that can carry solids, liquids, metal and plastic scraps, and almost any other material that can fit through a 7 inch hose. They are equipped with a boom and up to 500 feet of hose that allow them to convey materials from otherwise inaccessible areas. They are available in capacities ranging from 1,250 to 6,000 gallons. Their mobility and large capacity eliminate the need to transfer wastes to other vehicles before hauling for disposal or treatment. Vacuum loaders can operate in either a solids or liquids handling mode. Changing modes can be done quickly with an exterior adjustment and without emptying the load. This allows the Vactor or Supersucker to convey both soils and pools of liquid waste without dumping the load.

Portable skid-mounted vacuum units are also available. These can be airlifted, dragged by bulldozer, or even hauled on the back of a pickup truck to otherwise inaccessible areas. These units are generally available in capacities ranging from 1,900 to 5,700 liters (500-1,500 gallons), although units that can handle up to 11,400 liters (3,000 gallons) are manufactured. Skid-mounted units with vapor recovery systems are also available.

A number of factors should be considered prior to contracting for the services of a vacuum truck. Because of the large capacity of the vacuum cylinder, vacuum trucks are generally not well suited to sites with fewer than 30 drums to be consolidated. For a small site, it is generally more cost-effective to overpack the drums or to use a vacuum skid-mounted unit. This is due to high transportation costs and cost of handling wastewater generated from decontaminating the truck.

The cost of decontamination can be substantially reduced by a number of good management practices. The vacuum truck, or skid-mounted unit should be dedicated as much as possible to handling a certain type of waste so that decontamination is not required between each load. The units should also be sized for the job so that excessive decontamination water is not generated as a result of choosing an oversized vacuum cylinder.

Another important factor to consider in selecting vacuum trucks or skid-mounted units is the compatibility of wastes with materials of construction. Vacuum cylinders can be purchased in carbon steel, stainless steel, aluminum, and nickel. They can also be treated with a variety of coatings including epoxy, fiberglass, and neoprene rubber. In addition to selecting vacuum trucks with compatible liners, compatibility problems can be minimized by allowing wastes to react in a "reaction tank" or "compatibility chamber" where the heat of reaction can be released before pumping the wastes into the vacuum truck.

Costs--According to a representative of Peabody-Myers, a truck having a capacity of 3,000 gallons and production rate of 1,000 gallons/minute has a rental cost of $100/hr. This figure includes all costs except initial transportation to the job site (Peabody-Myers, personal communication, 1985).

7.1.4 Planning of Excavation and Removal Activities

There are a number of activities which are performed prior to and as part of excavation and removal activities. These include: design and construction of site operating areas; implementation of controls to minimize environmental releases and protect worker safety; and equipment selection and mobilization. Each is discussed briefly below.

7.1.4.1 Operational Layout

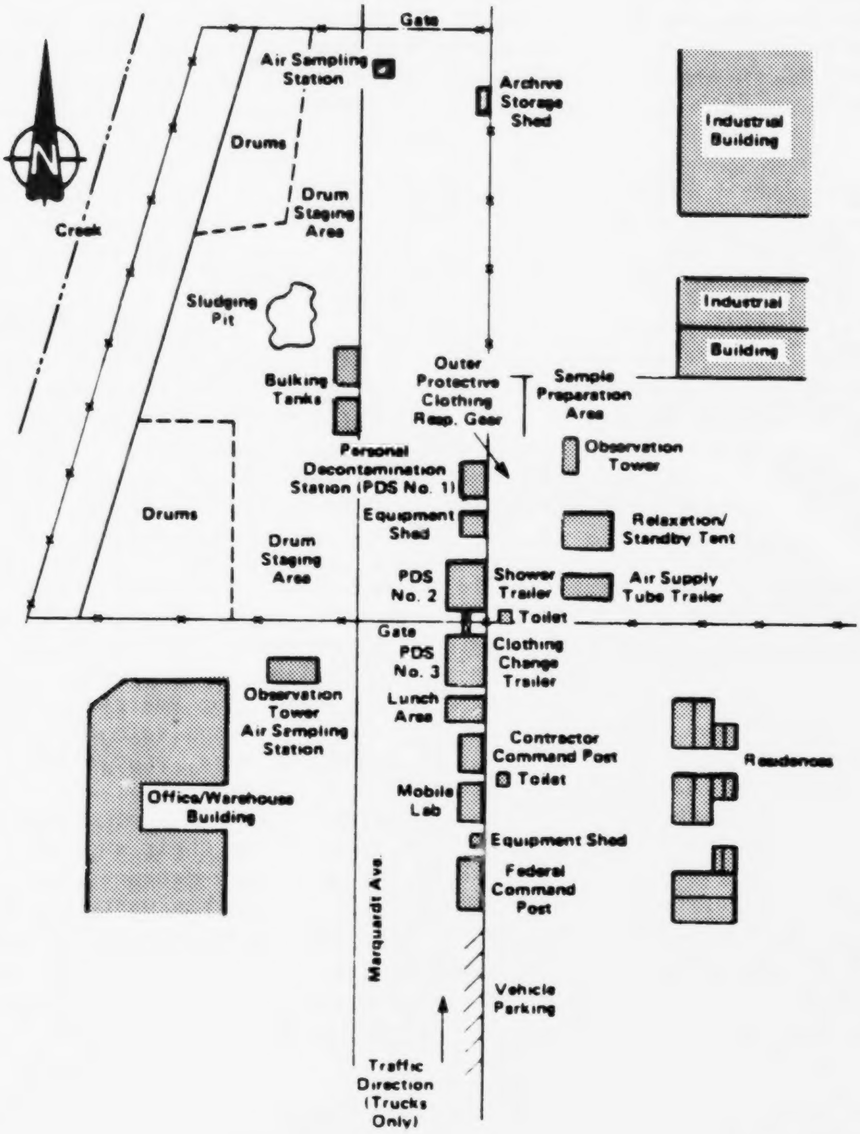
Proper layout of the work area (including support facilities) is critical to safe and cost-effective excavation and removal. Figure 7-4 illustrates structures, buildings, and operating areas for an example site. "Hot," "transition," and "clean zones" should be established using air monitoring data and available information on waste locations. The location of these zones should govern where activities are carried out. For example, any staging or on-site treatment of wastes would be conducted in the contaminated zone; personnel decontamination would be carried out in the transition zone; and administrative and emergency medical care would be carried out in the clean zone.

Distinct operating areas should be provided for staging, treating, storage and transport of wastes, and equipment decontamination. Figure 7-5 shows the operating areas for a waste site containing drums, contaminated soils, and lab packs. Each area should be designed such that there is adequate room to maneuver equipment and to provide for emergency evacuation. A careful evaluation is needed to determine the minimum safe distance between different operating areas. Within each operating area, provisions must be made to segregate reactive, corrosive, explosive, flammable, and incompatible wastes. Wastes which are explosive or radioactive should be staged or stored in isolated areas until arrangements can be made for their safe detonation or off-site disposal. For each operating area, necessary measures must be taken to minimize environmental releases, prevent incompatible waste reactions, and contain contaminants which are released. These measures are described in the following subsections.

7.1.4.2 Environmental Controls

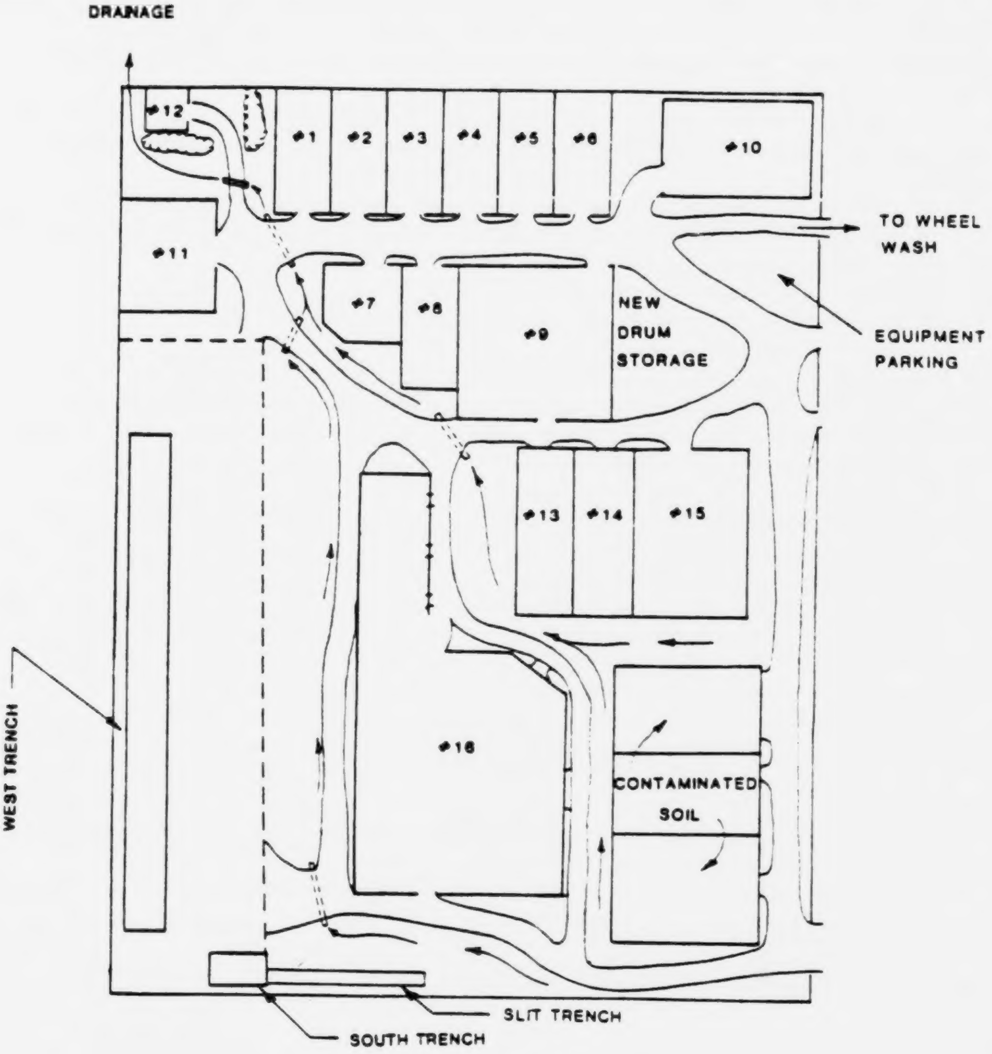
The nature and extent of preventive and mitigative measures required for controlling environmental releases during excavation and removal are site specific, although there are a number of general procedures that apply to all sites. Operating areas for staging and treating drummed wastes and contaminated soils should be graded to prevent puddling; lined with polyethylene or clay; and bermed or diked. (This design will provide only minimal secondary containment and will not be acceptable at many sites.) Where temporary impoundments must be used to store liquids, it may be acceptable to provide

FIGURE 7-4.
SITE OPERATIONAL LAYOUT



Source: Buecker and Bradford, 1982
(Manuscripts originally printed in Proceedings
of the National Conf. on Management of
Uncontrolled Hazardous Waste Sites, 1982.
Available from Hazardous Materials Control
Research Institute, 9300 Columbia Blvd.,
Silver Spring, MD 20910)

FIGURE 7-5.
PICILLO HAZARDOUS WASTE SITE LAYOUT (WESTERN TRENCH)



CELL	FUNCTION
1-8	SOLIDS STORAGE/MIXING
9	STAGING/SAMPLING
10	DRUM CRUSH, RESERVE STORAGE
11	LAB PACK STORAGE
12	LAB PACK DEMOLITION
13,14	ACID,PCB DRUM STORAGE
15	CONTAMINATED SOIL
16	LIQUID SAMPLE/STAGE/BULK

Source: Perkins Jordan, Inc., 1982

a thick clay liner and to excavate the contaminated soils after use of the impoundment is completed. Long storage periods or poor site conditions (e.g., wastes in the water table or permeable unsaturated zone) may necessitate the use of a synthetic liner system. The equipment decontamination area should be a hard surface area that will retain wash water by perimeter curbing and collect these liquids by means of a central trough and perimeter sump.

In addition to the above-mentioned preventive measures, a number of other measures may be taken to mitigate and minimize releases. Such measures include:

- Covering contaminated soils which have been excavated to prevent leaching of contaminants and fugitive dusts
- Using sorbents, pumps, or other equipment throughout the operation to clean up spills promptly
- Maintaining drums, overpacks, or other types of containers at strategic locations in work areas and on access roads to be used for prompt cleanup of spills
- Constructing surface water diversions around the site to control run-on and run-off
- Constructing a holding pond downslope of the site to contain contaminated run-off
- Avoiding uncontrolled mixing of incompatible wastes
- Promptly overpacking or transferring the contents of any drum that is leaking or may soon leak; promptly resealing drums following sampling
- Using sand, foams, etc. to suppress small fires before they spread
- Avoiding storage of explosives or reactive wastes in the vicinity of buildings or in confined areas
- Covering wastes that are known to be water reactive.

7.1.4.3 Health and Safety of Field Personnel

The USEPA and the Occupational Safety and Health Administration (OSHA) have published guidelines on health and safety procedures applicable to the cleanup of uncontrolled hazardous waste sites. These guidelines should be considered at all remedial action sites; however, they will not be covered in this report. A partial list of field health and safety documents are presented below:

- USEPA, Interim Standard Operating Safety Procedures, September 1982
- NIOSH, A Recommended Standard for Occupational Exposure to Hot Environments, HSM No. 72-10269
- OSHA, Code of Federal Regulations, Title 29, Section 1910.134
- American Industrial Hygiene Association, Respiratory Protection, A Manual and Guideline, AIHA, 1980
- American National Standards Institute, Inc., Practices for Respiratory Protection, ANSI Z-88.2-1980, New York, 1983, p. 3-5
- Federal Emergency Management Agency, Planning Guide and Checklist for Hazardous Materials Contingency Plans, FEMA-10, July 1981.

7.1.4.4 Equipment Selection/Mobilization

The choice of equipment for excavation/removal activities is based largely on inherent capabilities and limitations of the equipment. These factors were discussed in Section 7.1.3 and summarized in Table 7-3. Other factors affecting equipment selection include:

- Equipment efficiency under site-specific conditions
- Equipment dispatching time (transport and setup)
- Contractor performance record with equipment
- Equipment idle time
- Equipment versatility
- Equipment modifications to increase efficiency and safety
- Equipment capability for remote waste handling.

7.1.5 Excavation/Removal Procedures

Regardless of the types of equipment used for excavation and handling, certain standard operating procedures and safety practices should be followed.

Table 7-3

EXCAVATION/REMOVAL EQUIPMENT CAPABILITIES AND LIMITATIONS

Equipment Type	Excavation of Cover Materials	Excavation Depths	Precise Excavation Around Buried	Removal of Liquids	Hauling Capabilities	Limitations/Disadvantages
Backhoes (and attachments)	x	x	x			Maximum digging depth limited to 30 feet with conventional ventional backhoes. Maximum reach limited to 100 feet.
Dozers and loaders (and attachments)	x	x	x			Limited in speed and mobility in marshy and swampy areas. Have to be used in combination with other excavation equipment such as backhoes.
Cranes and attachments (clamshells, Draglines)	x	x	x			Difficult to spot bucket for scraping and dumping. Cannot backfill or compact. Very limited mobility; slow.
Scrapers	x				x	Excellent loading action; needs to be used simultaneously with other excavation equipment such as dozer and loaders.
Haulers					x	Used only for transport purposes and not for excavation.
Pumping system				x		Not well suited for removal of liquids with high solids content.
Industrial vacuums	x		x	x	x	Cost-effective only for large scale operations.

As soils are being excavated on-site, air monitoring should be conducted to determine unsafe levels of various hazardous constituents. Numerous portable direct reading instruments are available for this purpose. These include:

- Combustible gas detectors for measuring the lower explosive limit (lowest concentration of flammable gas that will explode, ignite or burn when there is an ignition source)
- Oxygen meters for measuring percent oxygen
- Photoionization detectors, flame ionization detectors, infrared analyzers and detector tubes for measuring gases and vapors
- Radiation meters, including alpha/beta/gamma survey meters, gamma radiation meters and dosimeters for detecting radiation

Where drums are present, a visual inspection should be made of the drum to determine whether it is empty, intact, leaking, or potentially dangerous; as evidenced by bulking, buckling, corrosion, and other deformations.

Drum identification and inventory should begin before excavation. Information such as location, date of removal, drum identification number, overpack status, and any other identification marks should be recorded on the drum inventory forms.

If there is an indication that the drums contain explosive or shock-sensitive materials, they should be handled remotely, or at a minimum, with vehicles equipped with plexiglas safety shields.

If a drum is critically overpressurized, it should be isolated with a barricade or steel demolition net until the pressure can be relieved remotely. A tarpaulin may also be used to cover the drum, provided the cloth is positioned remotely using long poles or rods. However, it must be cautioned that the mere weight of the tarpaulin or change in position of the drum could cause rupture. Slow venting using a bung wrench and plastic cover over the drum has worked for less critical situations; however, this should only be attempted by experienced personnel and extreme caution should be exercised.

Soils and drums containing ionizing levels of radiation should be handled on a site-specific basis. Generally, when such wastes are identified (via radiation meters), they are immediately drummed or overpacked using remotely operated equipment and moved to a separate staging area. However, depending on the level of radiation, special shielding devices may be required to protect field personnel. The Safety Officer should be consulted if radioactive materials are encountered.

Any drum that is leaking, badly corroded, or deformed should be overpacked or should have its contents transferred to a new or reconditioned drum. In some instances, it may be possible to transfer the drum contents to a "compatibility" chamber or vacuum truck. These procedures, however, are usually used for bulking after wastes have been identified rather than at this stage, since lack of knowledge about waste types could result in incompatible waste reactions.

If gas cylinders are encountered, they should be moved promptly to an area where the temperature can be controlled, particularly if they are subjected to temperature extremes or direct sunlight. Gas cylinders should not be rolled, dragged, or slid, even for short distances. Care should be taken not to drop the cylinders or allow them to violently strike another cylinder or drum (Matheson Gas Products, undated).

As contaminated soils are excavated from the disposal area, they should be transferred to box trucks or to a temporary storage area, preferably a diked or bermed area lined with plastic or low permeability clay. A layer of absorbent material should be placed on the bottom of the temporary storage area. Frequently, gas analyzers are used to determine the approximate level of contamination of soils. Soils can then be segregated based on contaminant levels. Pools of liquid wastes should be promptly removed using pumps or solvent materials.

7.2 Off-Site Disposal

7.2.1 General Description

This section describes the major factors that must be considered in selection of an off-site disposal facility and preparing wastes for off-site transport. Off-site disposal, as described in this section, includes landfilling and incineration. Off-site waste treatment is described in Section 10.

7.2.2 Applications/Limitations

Determining the feasibility of off-site disposal requires knowledge of RCRA regulations (40 CFR Parts 261-265) and other regulations developed by State Governments. RCRA manifest requirements, under 40 CFR Parts 262 and 263, must be complied with for all wastes that are shipped off-site. In addition, the waste generator (or other responsible party, when the generator is unknown) must comply with RCRA manifest requirements under 40 CFR Parts 262 and 263, and the generator should comply with applicable hazardous waste generator requirements under 40 CFR Part 262. In addition, the generator should ensure that the facility selected to receive the wastes is in compliance with all applicable Federal and State environment and public health statutes. Under 40 CFR 264.12, RCRA storage, and disposal facilities are required to notify the generator, in writing, that they are capable of managing the wastes. The generator must keep a copy of this written notification on file as part of the operating record.

7.2.2.1 Off-Site Landfilling

Landfilling of hazardous materials is becoming increasingly difficult and more expensive due to steadily growing regulatory control of this technology. Therefore, wastes that are amenable to treatment or incineration should be segregated from wastes for which no treatment alternative is known. Landfilling should usually be regarded as the least attractive alternative at a site cleanup action.

Landfilling costs are approximately \$240/ton for highly toxic wastes (e.g., high levels of chlorinated hydrocarbons), \$120/ton for ignitable materials (40 CFR, Part 264.312 must be complied with), \$85/ton for most industrial sludges, and \$40 to \$50/ton for municipal treatment sludges.

7.2.2.2 Incineration

Among the most important factors which a treatment facility considers in determining the suitability of wastes for incineration include: btu content of the waste, viscosity, water content, halogen content and ash content. Many incineration facilities' permit conditions specify a minimum acceptable btu content. The minimum acceptable btu is generally not less than about 5000 btu/lb since at this heating value the incineration process can often be self sustaining and require no auxiliary fuel. High water content in a waste tends to reduce the heating value and therefore many facilities will specify a maximum allowable water content. Certain incinerators are not equipped to handle highly viscous or solid wastes, and this may be yet another criteria which they use to accept or reject a waste load.

EPA regulations (under RCRA) for hazardous waste incineration require that particulate emissions be no more than 180 mg/Nm^3 and that hydrogen chloride removal efficiency from the exhaust gas be no less than 99 percent. Trial burns are conducted prior to issuance of a permit to determine the maximum ash and chlorine content which a waste can handle in order to meet these requirements. Thus, the facility is likely to have maximum limits for halogen content and ash. Incineration of PCB's and low-level radioactive wastes requires special permits and there are only a limited number of facilities permitted to handle these wastes.

7.2.3 Implementation

7.2.3.1 Preparation of Wastes for Off-Site Treatment/Disposal

Where drums or multiple impoundments are present it is often most cost-effective to consolidate their contents in a tank truck. Compatibility testing should be performed prior to bulking wastes for off-site transport to ensure that consolidation will not result in incompatible waste reactions or in large volumes of waste which are unacceptable for off-site disposal. Compatibility testing refers to simple, rapid, and cost-effective testing procedures that are used to segregate wastes into broad categories. By identifying broad waste categories, compatible waste types can be safely bulked on-site without risk of fire, explosion, or release of toxic gases, and disposal options can be determined without an exhaustive analysis of each waste type.

Compatibility testing protocols have been developed by a number of cleanup contractors and generators. Often, however, the procedures must be tailored to meet the testing requirements of prospective treatment/disposal facilities. The Chemical Manufacturers Assocation (CMA, 1982) has developed a compatibility testing protocol which has been used at a number of sites. Based on the CMA protocol, wastes can be segregated into the following broad waste categories:

- Liquids
  - Radioactive
  - Peroxides and oxidizing agents
  - Reducing agents
  - Water-reactive compounds
- Water insolubles
  - Low halogen, low PCB
  - Mixed halogen, high PCB
  - High halogen, low PCB
- Acids
  - Strong (pH <2)
  - Weak (pH 7-12), with or without cyanides or sulfides
- Bases
  - Strong (pH >12), with and without cyanides or sulfides
  - Weak (pH 7-12), with and without cyanides or sulfides
- Solids
  - Radioactive
  - Nonradioactive.

Testing to determine gross halogen content is sometimes eliminated if all insoluble wastes are to be incinerated at a facility capable of handling chlorinated organics. However, testing for PCBs is required regardless of the need for testing other halogenated compounds.

The CMA protocol also requires that small samples of wastes that are intended to be bulked are mixed together. Visual observations are then made for precipitation, temperature changes, or phase separation.

There are some differences between the CMA compatibility protocol and the protocol used by some cleanup contractors. One commonly used procedure is to conduct flammability and ignitability tests on a drum-by-drum or waste-by-waste basis for both liquid and solid drums. CMA, on the other hand, recommends that these tests be performed on composite samples before bulking, since these tests require more costly and time-consuming analysis (torch test and closed cup flame test). Another common practice is to conduct further testing on samples from drums containing solids. These tests may include water reactivity, water solubility, pH, and the presence of oxidizers. In general, the decision to perform these analyses on each waste rather than on a composite sample (prior to bulking) is made based on the number of drums and the types of wastes known to be present on site.

Hatayama et al. (1980a, 1980b) have also provided guidance on waste incompatibilities that can be useful during the waste consolidation process. These researchers have developed a hazardous waste compatibility protocol that allows the user to evaluate potential adverse reactions for binary combinations of hazardous wastes. Binary waste combinations are evaluated in terms of the following adverse reactions: heat generation from a chemical reaction, fire, toxic gas generation, flammable gas generation, explosion, and violent polymerization of toxic substance.

A detailed waste analysis is generally required before a waste is accepted by a treatment/disposal facility. Table 7-4 specifies the types of analyses which are typically required before a waste can be considered for off-site disposal at a particular facility. However, requirements vary considerably depending upon the facility permits, state regulations, physical state of the wastes, and the final disposal option which is selected.

On-site pretreatment of wastes may be required in order to make them acceptable for off-site transport or to meet the requirements of an incineration or disposal facility. For incineration or land disposal facilities, pretreatment will likely be limited to the following:

- Acid-base neutralization (land disposal and incineration)
- Metal precipitation/solidification (land disposal)
- Hypochlorite oxidation of cyanide and sulfide (land disposal and incineration)
- Flash point reduction (land disposal)
- Removal of free liquids by addition of soils, lime, fly ash, polymers, or other materials which remove free water (land disposal).

TABLE 7-4.
POTENTIAL ANALYTICAL REQUIREMENTS FOR DISPOSAL

PHYSICAL STATE

- Physical state at 70°F
- Number of layers
- Free liquids (percent by volume)
- pH
- Specific gravity
- Flash point
- Viscosity

WASTE COMPOSITION

- EP Toxic Metals (Arsenic, Barium, Cadmium, Chromium, Lead, Mercury, Selenium, Silver)
- EP Toxic Pesticides (Endrin, Lindane, Methoxychlor Toxophene, 2,4-D, 2,4,5-TP)

- Hydrocarbon Composition (must account for 100 percent)

- Organochlorine

- Sulfur

- Cyanide

- PCB content

HAZARDOUS CHARACTERISTICS

- Reactive (pyrophoric or shock sensitive)
- Explosive
- Water reactive
- Radioactive
- Ignitable

7.2.3.2 Transportation

The transportation of hazardous wastes is regulated by the Department of Transportation, the Environmental Protection Agency, the States, and, in some instances, by local ordinances and codes. In addition, more stringent federal regulations also govern the transportation and disposal of highly toxic and hazardous materials such as PCBs and radioactive wastes. Applicable Department of Transportation regulations include:

- Department of Transportation 49 CFR, Parts 172-179
- Department of Transportation 49 CFR, Part 1387 (46 FR 30974, 47073)
- Department of Transportation DOT-E 8876.

The USEPA regulations under RCRA (40 CFR Parts 262 and 263) adopt DOT regulations pertaining to labelling, placarding, packaging, and spill reporting. These regulations also impose certain additional requirements for compliance with the manifest system and recordkeeping.

Vehicles for off-site transport of hazardous wastes must be DOT approved and must display the proper DOT placard. Liquid wastes must be hauled in tanker trucks that meet certain requirements and specifications for the waste types. Contaminated soils are hauled in box trailers and drums in box trailers or flat bed trucks. The trucks should be lined with plastic and/or absorbent materials.

Before a vehicle is allowed to leave the site, it should be rinsed or scrubbed to remove contaminants. Both bulk liquid containers and box trailers should be checked for proper placarding, cleanliness, tractor-to-trailer hitch, and excess waste levels. Bulk liquid containers should also be checked for proper venting, closed valve positions, and secured hatches. Box trailers should be checked to ensure correct liner installation, secured cover tarpaulin, and locked lift gate.

7.2.4 Selection/Evaluation Considerations

Excavation and removal can almost totally eliminate the contamination at a site and the need for long-term monitoring. Once excavation is begun, the time to achieve beneficial results can be short relative to such alternatives as in-situ treatment, subsurface drains, and, in some instances, pumping. Excavation and removal can be used in combination with almost any other remedial technologies.

The biggest drawbacks with excavation, removal, and off-site disposal are associated with worker safety, short term impacts, cost, and institutional aspects. Where highly hazardous or toxic materials are present, excavation can pose a substantial risk to worker safety. Short term impacts such as fugitive dust emissions, toxic gases, and contaminated run-off are frequently a major concern, although mitigative measures can be taken. Costs associated with off-site disposal are high and frequently result in exclusion of complete excavation and removal as a cost-effective alternative. The location of a RCRA approved landfill or incinerator also has a substantial impact on costs.

7.3 On-site Land Disposal

7.3.1 General Description

This section describes on-site disposal of wastes by landfilling. It involves the design and construction of new landfills which comply substantially with RCRA landfill facility standards under 40 CFR Part 264. It should be noted that EPA guidance for CERCLA responses requires most on-site disposal actions "to attain or exceed applicable and relevant standards of [all] Federal public health and environmental laws, unless specific circumstances" dictate otherwise (FR 29:5862-5932, 1985). However, permitting requirements under the laws are usually not required for fund-financed or enforcement actions under CERCLA (FR 29:5862-5932, 1985).

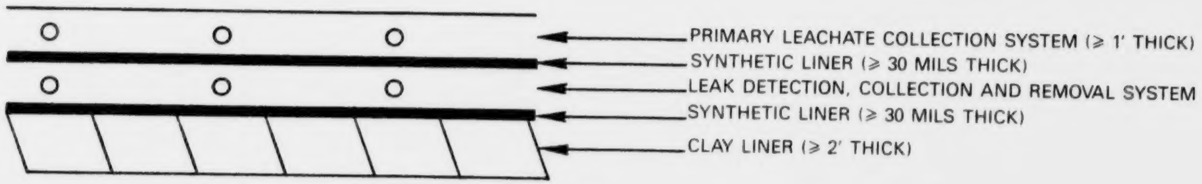
7.3.2 Application/Limitations

The RCRA requirements under 40 CFR Part 264 and all associated guidance are concerned with the proper location, design, construction, operation, and maintenance of hazardous waste management facilities. These requirements preclude landfilling in areas of seismic instability, in a 100-year floodplain, and where the integrity of the liner system would be adversely affected. These requirements also preclude landfilling of liquids and several types of highly mobile and/or highly toxic wastes. In addition to complying with these requirements, the evaluation of an on-site landfill program must address potential risks posed by the depth to groundwater at the site and the degree of naturally available groundwater protection if the liner system should fail. Other factors entering this evaluation include costs for monitoring the groundwater, collecting any accumulated leachate, and for implementing further corrective action if the groundwater has been contaminated by a leak from the new landfill. Furthermore, it may be technically infeasible to develop a groundwater monitoring program at sites where the groundwater has already been contaminated.

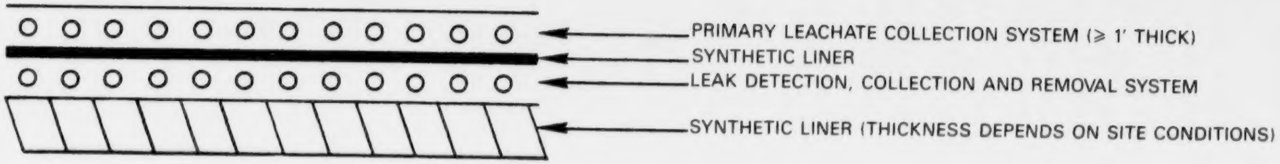
7.3.3 Landfill System Design, Construction, and Implementation

This section describes the design of a landfill with a double liner system and two leachate detection, collection, and removal systems according to applicable RCRA requirements. Figure 7-6 shows a diagram which illustrates the two double liner designs described in EPA guidance. One of these designs involves two synthetic liners and at least 5 feet of clay; the other involves one synthetic liner and a clay layer that will not be penetrated by waste leachate for at least 30 years, even if the synthetic liner fails (USEPA, 1984).

FIGURE 7-6.
SCHEMATIC DIAGRAM OF TWO DOUBLE LINER DESIGNS FOR LANDFILLS


PRIMARY LEACHATE COLLECTION SYSTEM (≥ 1' THICK)
SYNTHETIC LINER (≥ 30 MILS THICK)
LEAK DETECTION, COLLECTION AND REMOVAL SYSTEM
SYNTHETIC LINER (≥ 30 MILS THICK)
CLAY LINER (≥ 2' THICK)

DESIGN 1


PRIMARY LEACHATE COLLECTION SYSTEM (≥ 1' THICK)
SYNTHETIC LINER
LEAK DETECTION, COLLECTION AND REMOVAL SYSTEM
SYNTHETIC LINER (THICKNESS DEPENDS ON SITE CONDITIONS)

DESIGN 2

7.3.3.1 Overall Liner System Design Standards

Specific RCRA requirements pertaining to hazardous waste landfill liner systems include the following (Cope et al., 1984):

- Liner materials must be compatible with the waste material being disposed and must be able to withstand any physical forces such as hydrostatic pressure, adverse climatic changes, and other physical stresses, such as the stresses of installation and the heavy equipment used to move and compact wastes.
- Liners must be placed on a stable foundation designed to prevent failure due to settlement, compression, uplifting, or warping likely to be caused by unexpected changes in pressure gradients above, below, or adjacent to the liner material.
- All liners must be installed to ensure that the waste or leachate cannot come into contact with the surrounding soils.
- The liner system must be monitored and inspected during construction and installation, and inspected for uniformity, damage, and imperfections following installation. Soil-based and admixed liners and covers must be inspected for imperfections including lenses, cracks, channels, root holes, or other structural nonuniformities that may cause an increase in the permeability of the liner or cover.

Evaluation of the site's geological and hydrological conditions is critical to developing a well engineered hazardous waste landfill. Table 7-5 summarizes the major adverse site conditions which can result in liner failure. Preventive measures must be taken to prevent liner failure under these conditions.

7.3.3.2 Primary Leachate Collection and Removal System

RCRA requirements mandate that leachate collection and removal systems be placed immediately above the primary liner in all new hazardous waste landfills. Such systems must be capable of maintaining a leachate depth of 1 foot or less above the liner and withstanding clogging, chemical attack, and forces exerted by wastes, equipment, or soil cover.

EPA guidance documents recommend that the leachate collection system consist of a drainage layer at least 1 foot thick, with a hydraulic conductivity $\geq 1 \times 10^{-3}$ cm/sec, and a minimum slope of 2 percent. When installed over a secondary clay liner with hydraulic conductivity of 1×10^{-7} cm/sec, such a system provides the four-order-of-magnitude difference in permeability known to significantly increase drainage efficiency (Cope et al., 1984). The drainage layer should be covered by a filter (graded sand layer or geotextiles) to prevent infiltration of fines from the waste and subsequent clogging of the drainage layer.

TABLE 7-5.
SUMMARY OF ADVERSE SITE CONDITIONS AFFECTING LINER PERFORMANCE

Unfavorable Condition	Potential Liner Problem
• Geotechnical/Hydrogeologic	
Moderate to active seismic area	Levee instability; liner failure
Settlement or subsidence	Cracks in clay or tears in synthetic liners
High groundwater table	Lifting or rupturing of liner
Voids	Cracking of liner
Sinkholes	Liner failure
Subsurface gas	Lifting of liner prior to backfilling
High permeability soils	Piping of subgrade
• Climatic	
Frozen ground/ice	Cracking, tearing
Wind	Lifting and tearing liner
Sunlight	Dehydration of clay liner (permitting cracks to develop)
	Destruction of some synthetic liners (caused by ultraviolet radiation)
High humidity	Poor seam adhesion caused by absorption of moisture by the solvents

Source: Cope et al., 1984

Leachate collection pipe networks should consist of slotted or perforated drain pipe bedded and backfilled with a gravel envelope. Layouts should include base liner slopes $\geq$ 2 percent and pipe grades $\geq$ 0.005. Pipe spacing should be determined for the unit (see Section 5.2). All pipes should be joined and, where appropriate, bonded (Cope et al., 1984). Sumps or basins should be installed at low points on the base of the fill to collect leachate discharging from the collection network. A riser pipe extending from the sump to the ground surface enables leachate removal.

7.3.3.3 Primary Synthetic Liner

A variety of synthetic and natural materials are available for use as liners. While soil liners are suitable for use as secondary liners, synthetic membranes are the primary mechanisms for long term containment. Table 7-6 summarizes the major characteristics, advantages, and disadvantages of various liner materials.

A critical first step in designing a liner system for a hazardous waste landfill is an evaluation of the physical and chemical composition of the wastes to be contained within the facility. Since the primary purpose of a liner is to prevent liquids from leaving a hazardous waste facility, the physical integrity of the liner and its chemical compatibility with the waste constituents must be ensured. A test method accepted by the USEPA for evaluating waste synthetic liner compatibility is presented in Appendix B of RCRA guidance document Landfill Design-Liner Systems and Final Cover. The method basically involves exposing a liner sample to the waste or leachate encountered at the facility and, after exposure, the sample is tested for strength and weight loss. Significant deterioration in these properties is considered evidence of incompatibility unless otherwise demonstrated (Cope et al., 1984).

Once a synthetic liner is selected (based on the criteria described earlier), the major focus of the design activities is on preparing a firm and smooth base for the membrane by compacting, scraping, and rolling the base. The major concerns during the installation of a synthetic membrane liner are providing protective soil layers above and below the liner and proper seaming of the liner. This requires that manufacturers' installation procedures and practices be followed for the specific type of membrane proposed. Each type of membrane liner also requires specific seaming provisions to ensure an effective bond. Since adverse weather conditions (e.g., extreme heat or cold, precipitation, winds) can affect adequate bonding of the liner field seams, installation should be avoided during these periods, unless protective measures are used (Cope et al., 1984).

During placement of the liner and before wastes are placed in the landfill, tests of seam strength and bonding effectiveness should be conducted. In addition, random samples of seams should be cut from the liner and subjected to on-site and laboratory testing. Liner placement, seaming, and testing are covered in detail in the USEPA technical document Lining of Waste Impoundments and Disposal Facilities, SW-870 (Cope et al., 1984).

TABLE 7-6.
CHARACTERISTICS, ADVANTAGES, AND DISADVANTAGES OF SELECTED SYNTHETIC LINERS

Liner Material	Characteristics	Range of Costs	Advantages	Disadvantages
Butyl rubber	Copolymer of isobutylene with small amounts of isoprene	M	Low gas and water vapor permeability; thermal stability; only slightly affected by oxygenated solvents and other polar liquids	Highly swollen by hydrocarbon solvents and petroleum oils; difficult to seam and repair
Chlorinated polyethylene	Produced by chemical reaction between chlorine and high density polyethylene	M M	Good tensile strength and elongation strength; resistant to many inorganics	Will swell in presence of aromatic hydrocarbons and oils; high elongation, poor memory
Chlorosulfonated polyethylene	Family of polymers prepared by reacting polyethylene with chlorine and sulfur dioxide	M	Good resistance to ozone, heat, acids, and alkalis; easy to seam	Tensile strength increases on aging; good tensile strength when supported; poor resistance to oil
Epichlorohydrin rubbers	Saturated high molecular weight, aliphatic polyethers with chloromethyl side chains	H	Good tensile and tear strength; thermal stability; low rate of gas and vapor permeability; weathering; resistant to hydrocarbons, solvents, fuels, and oils	Difficult to field seam or repair
Ethylene propylene rubber	Family of terpolymers of ethylene, propylene, and nonconjugated hydrocarbon	M	Resistant to dilute concentrations of acids, alkalis, silicates, phosphates and brine; tolerates extreme temperatures; flexible at low temperatures; excellent resistance to weather and ultraviolet exposure	Not recommended for petroleum solvents or halogenated solents; difficult to seam or repair; low seam strength
Neoprene	Synthetic rubber based on chloroprene	H	Resistant to oils, weathering, ozone and ultraviolet radiation; resistant to puncture, abrasion, and mechanical damage	Difficult to seam or repair
Polyvinyl chloride	Produced in roll form in various widths and thicknesses; polymerization of vinyl chloride monomer	L	Good resistance to inorganics; good tensile, elongation, puncture, and abrasion resistant properties; wide ranges of physical properties; easy to seam	Attacked by many organics, including hydrocarbons, solvents and oils; not recommended for exposure to weathering and ultraviolet light conditions
Thermoplastic elastomers	Relatively new class of polymeric materials ranging from highly polar to nonpolar	M	Excellent oil, fuel, and water resistance with high tensile strength and excellent resistance to weathering and ozone	None reported
High Density Polyethylene	Blow or sheet extended P.E.	M to H (based on thickness)	Good resistance to oils and chemicals; resistant to weathering; available in 20 to 150 mils thicknesses; resistance to high temperature	Thicker sheets require more field seams; subject to stress cracking; subject to puncture at lower thicknesses. Poor tear propagation.

1. Adapted from Technologies and Management Strategies for Hazardous Waste Control, Office of Technology Assessment, Congress of the U.S., 1983. Modified in consultation with industry experts.
2. Cost ranges: $L = \$1\text{-}4/\text{yd}^2$, $M = \$4\text{-}8/\text{yd}^2$, $H = \$8\text{-}12\text{yd}^2$ (installed costs).
3. All ratings are based on properly compounded materials designed for that specific application.

Source: Cope et al., 1984

7.3.3.4 Secondary Leachate Collection

The secondary leachate collection system is located between the two liners and is generally used to detect and remove any liquid which could migrate into the space separating the liners. It is designed similarly to the primary leachate collection and removal system in which liquid is collected in a porous medium and subsequently removed by gravity using a network of perforated pipes.

7.3.3.5 Secondary Liner System

The secondary liner should have an "adequate thickness to avoid failure throughout the post-closure monitoring period, and be chemically resistant to the waste and leachate managed at the landfill" (USEPA, 1984). Soil liners may be constructed of native clay materials exhibiting a remolded permeability of $1 \leq \times 10^{-7}$ cm/sec and obtained either on-site, from selected borrow areas, or from off-site sources. Proper installation of a soil liner is needed to maintain this specified minimum permeability (Cope et al., 1984). To ensure adequate compaction, soil materials should be spread in loose lifts no more than 6 inches deep, wetted or dried to 2 percent or more above optimum moisture content, and compacted with a sheepsfoot-type roller to the specified relative compaction. Specified values must be based upon the tested relationships between moisture content, relative compaction, and permeability. Furthermore, installation of a clay liner should not be attempted under adverse weather conditions such as heavy precipitation or freezing temperatures (Cope et al., 1984).

7.3.4 On-site Landfill Operation, Monitoring, and Maintenance

7.3.4.1 Operation and Maintenance

The operating life of an on-site landfill should be minimized to avoid unnecessary generation of leachate caused by rainfall into an open cell. Sometimes it is more efficient to construct several landfill cells in sequence rather than to construct one large cell which will remain open for a long time period. This determination is made by calculating and comparing the marginal costs of extra leachate treatment with the marginal costs of extra liner materials. During the operating life of an on-site landfill, every effort should be made to prevent run-on of rainwater into the landfill and to prevent hazardous liquid disposal. All liquids should be solidified prior to landfilling and the sorbent materials used should be resistant to later releases of liquids through biodegradation or pressure of overlying wastes. All materials placed into an on-site landfill should be compacted as much as possible using heavy equipment such as a sheepsfoot roller or a specially designed compactor vehicle. This practice will minimize settling after closure, and thus will protect the integrity of the final cap (see Section 3.1). All equipment and operators and other workers at the landfill should be thoroughly trained in the handling of all wastes to be placed in the landfill. These persons should also be trained in the proper operation of heavy equipment, emergency response, and the proper use of their safety equipment. During the operating life and post-closure period of the landfill, regular inspections should be conducted to check the integrity of the storm water controls, leachate collection/detetion sumps, and ground-water monitoring wells. Any broken equipment should be repaired or replaced immediately, and any leachate in the collection or detection sumps should be removed immediately. Maintenance procedures at a landfill are identical to those presented for capping (Section 3.1.3.5).

7.3.4.2 Groundwater Monitoring

RCRA requires all owners and operators of land disposal facilities to establish a groundwater monitoring program (Subpart F, Part 264). The groundwater monitoring program must be capable of determining the facility's impact on the quality of groundwater in the uppermost aquifer underlying the facility. In some cases it may be appropriate to monitor other hydrogeologic limits in addition to, or in lieu of, the uppermost aquifer, depending on specific conditions.

The minimum requirements for any groundwater monitoring system involve at least one upgradient well which is capable of yielding representative background samples and at least three downgradient wells whose location and depth ensure immediate detection of any statistically significant amounts of hazardous wastes or constituents in the upper aquifer. Where these minimum requirements do not allow the owner or operator to meet the overall performance objectives, he must determine where and how many additional wells are needed. Once established, groundwater monitoring programs must continue for an average of 30 years depending on site-specific conditions. During this period, groundwater samples are generally taken semiannually and analyzed for indicator parameters which are developed on a site-specific basis. Concentrations of indicator parameters from samples collected at the downgradient wells are individually compared to average background concentrations established from the upgradient well(s).

7.3.5 Technology Selection/Evaluation

On-site landfilling is an expensive technology which should only be considered when: (1) there is so much waste to be disposed that the total cost of off-site waste management at an acceptable site is comparable; (2) simple capping of the site will not provide adequate protection of human health and the environment; and (3) on-site conditions will allow the construction of a landfill that will protect human health and the environment. Since it is rare that all three of the above conditions are met at a site, the on-site landfill option is not frequently used.

7.3.6 Costs

The costs of constructing an on-site landfill depends on the size of the facility, the design features of each layer of the liner system, and site-specific engineering factors. However, general material and installation costs for the various components of a landfill are presented in Table 3-1 and Table 5-10. Excavation costs are presented in Section 7.1.3. The costs for monitoring well construction, sampling, and sample analyses can vary by orders of magnitude because of the number of factors involved. These factors include well depth, diameter, type of drill rig, type of substrate, types of wastes in the landfill, number of indicator parameters, level of indicator parameters, number of wells, levels of existing groundwater contamination, and area of the country. Therefore, the costs for groundwater monitoring programs cannot be estimated until a groundwater investigation has been conducted and a plan has been developed.

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SECTION 8

REMOVAL AND CONTAINMENT OF CONTAMINATED SEDIMENTS

Contamination of the bottom sediments of streams, ponds, lakes, harbors, estuaries, and other water bodies may occur via several different pathways. Contaminated soil may be eroded from the surface of hazardous waste disposal sites by natural runoff and be deposited in nearby watercourses or sediment basins constructed downslope of the site. Existing sediments along stream bottoms may absorb chemical pollutants that have been washed into the water course from disposal areas within the drainage basin. Similarly, contaminated groundwater may drain to surface water courses and the transported pollutants may settle into or chemically bind with the bottom sediments. Another source of sediment contamination results from spills of hazardous chemicals which are denser than water, sink to the bottom, and mix with and coat the sediments.

Remedial techniques for contaminated sediments generally involve sediment removal and subsequent treatment and disposal. During the removal of contaminated sediments, it is important to minimize the threat of further environmental harm through resuspension of contaminants. Various techniques have been developed to control this problem. Another aspect of the removal process involves measures for temporary diversion of water flow. Measures such as containment and in-situ treatment offer an alternative to removing the sediment. However, these measures are not well demonstrated.

8.1 Sediment Removal

The process of removing bottom sediments from a water body is commonly known as dredging. This process has been used for many years to widen or deepen harbors and navigable waters. In recent years, dredging has been employed in the removal of sediments that have been contaminated by hazardous substances.

Sections 8.1.1 through 8.1.3 describe the equipment used in the removal of contaminated sediments including mechanical, hydraulic, and pneumatic dredging. Section 8.1.4 addresses design and implementation considerations including information on predredging operations, dredge vessel control, and turbidity control. Dredging equipment is summarized in Section 8.1.5 in terms of technical feasibility and impacts. Costs of dredging are addressed in Section 8.1.6. Treatment of contaminated sediments generated during dredging operations is addressed in Sections 10.2 and 8.2.

8.1.1 Mechanical Dredging

Mechanical dredging involves the use of excavation equipment such as backhoes, draglines, clamshells, and bucket ladder dredges. Draglines and clamshells used for dredging are usually vessel-mounted, but can also be track-mounted and land-based.

The main advantage of mechanical dredging is removal of sediments at nearly in-situ densities, therefore maximizing solids content and minimizing the scale of facilities required for dredged material transport, treatment, and disposal. On the other hand, because mechanical dredging removes bottom sediment through direct application of mechanical force to dislodge the material, sediment resuspension (and therefore turbidity) is often high. Also, mechanical dredging is relatively ineffective in the removal of free or unabsorbed liquid contaminants. Additionally, mechanical dredging has a characteristically low production rate.

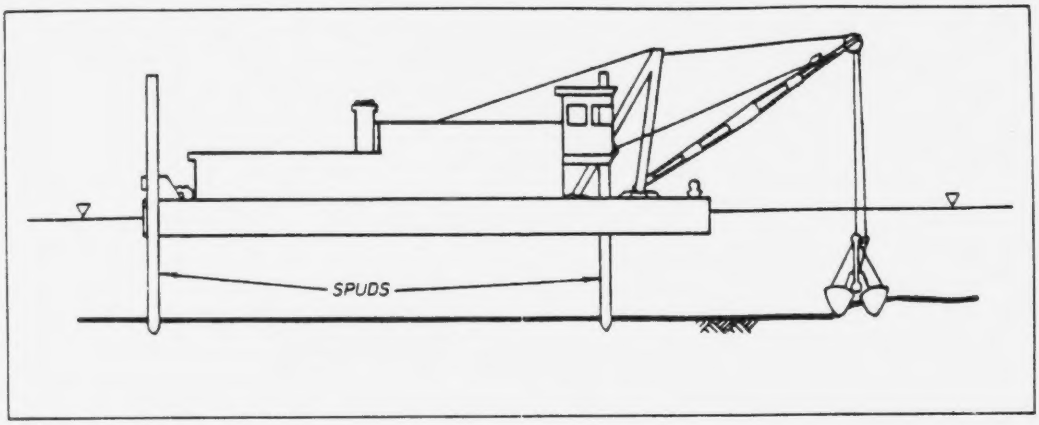
Mechanical dredging generally has application in streams and rivers that are relatively shallow and whose flow velocities are relatively low. It is also used for removing contaminated sediments deposited on dry river banks or in floodplains.

8.1.1.1 Clamshell Dredge

Clamshell (or grab) dredges are crane-operated devices. Most are equipped with one crane, but multiple crane configurations are not uncommon. The crane is normally mounted on a flat-bottomed barge or pontoon, but may also be crawler-mounted. Figure 8-1 illustrates a clamshell dredge.

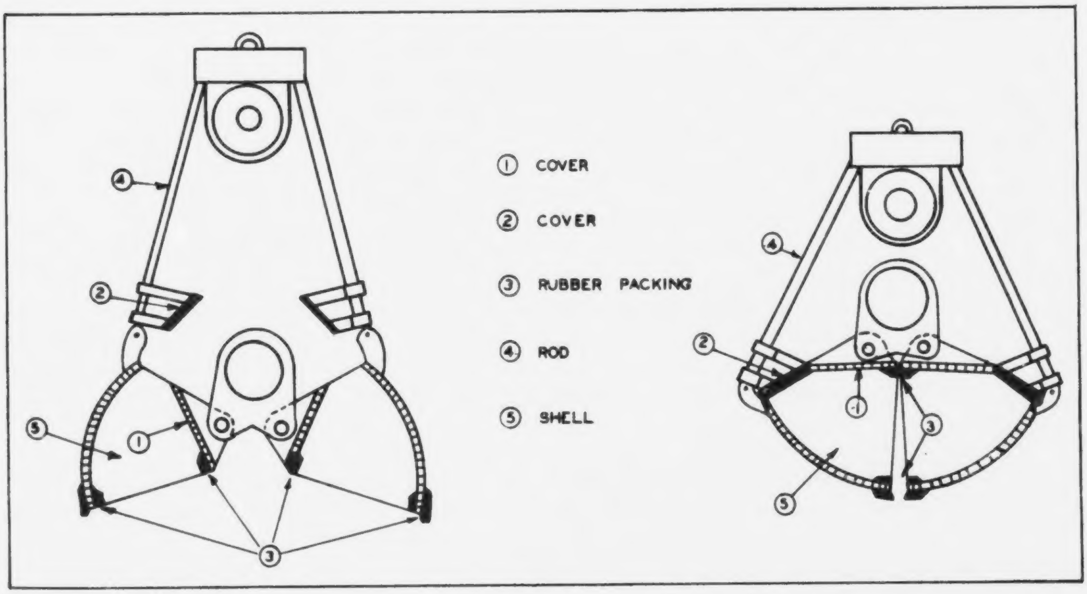
Production rates for clamshell dredges are relatively low, typically in the range of twenty to thirty cycles per hour. This varies with depth and working media. The working depth of the clamshell is limited, theoretically, only by the length of the cable. In practice, most clamshell dredges operate at depths of up to 100 feet. Clamshell buckets range in capacity from 1 to 12 cubic yards. These dredges excavate a heaped bucket of material, part of which is washed away by drag forces during hoisting. Once the bucket clears the water surface, additional losses may occur through rapid drainage of entrapped water and slumping of the material heaped above the rim (Hand, 1978). Loss of material is also influenced by the fit and condition of the bucket, the hoisting speed, and the properties of the sediment. Even under ideal conditions, substantial losses of loose and fine sediment will usually occur. To reduce the probability of fines and loose material escaping from the bucket during the hoisting operation, the Port and Harbor Institute of Japan has designed and fabricated a bucket that is completely closed and sealed by flexible gaskets, so that the dredged material is better contained within the bucket. This design is illustrated in Figure 8-2. A direct comparison of a standard open clamshell bucket with a watertight clamshell bucket indicates that watertight buckets generate 30 to 70 percent less resuspension in the water column than open buckets (Barnard, 1978). This

FIGURE 8-1.
CLAMSHELL DREDGE



Source: Hand et al., 1978

FIGURE 8-2.
OPEN AND CLOSED POSITIONS OF THE WATERTIGHT BUCKET



Source: Barnard, 1978

design has not been used in the United States, but similar modifications have been tested here.

A major criticism of the watertight clamshell is that the gaskets would not stand up to continuous use in a full-scale dredging operation, limiting its use to soft material and trash-free areas. In addition, compatibility problems may occur in certain chemical spill situations. Current design concepts include the use of an interlocking tongue-and-groove edge to overcome the sealing problems (Barnard, 1978 and Raymond, 1983).

a. Applications/Limitations

Clamshell dredges are adaptable to either land-based or barge-mounted operation. They are capable of excavating materials at nearly in-situ densities, and of excavating almost any type of material except the most cohesive consolidated sediment and solid rock. Clamshell dredges are easily controlled and maneuvered in small and very confined areas, and are capable of deep-water excavation. Availability of clamshell dredges is relatively good in the United States. Clamshell dredges are the subject of research and development to improve production efficiency, accuracy, and control, and to reduce adverse environmental effects.

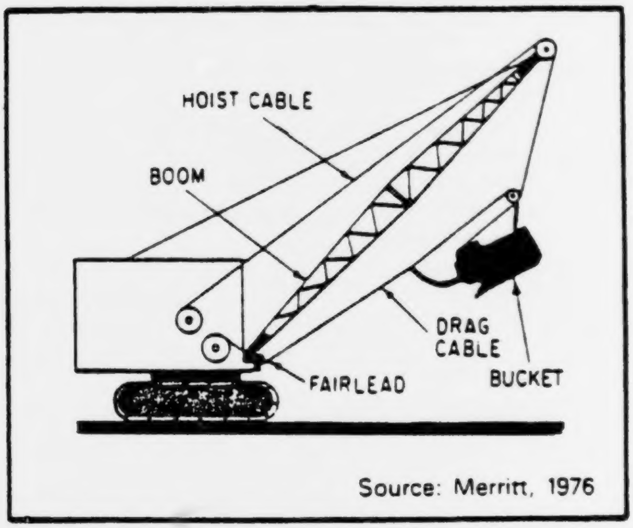
One of the greatest problems with clamshell dredges is that they generate a great deal of sediment resuspension and therefore turbidity. In addition, production rate is low. These dredges are inefficient in terms of losing material from the bucket during the hoisting operation. Clamshell dredges require separate disposal vessels and equipment for operation. Operation is limited to use in shallow streams and rivers with low flow velocity. High energy costs are associated with clamshell dredges, and maintenance costs are also high because of the clamshell's complex machinery (Hand, 1978).

8.1.1.2 Dragline Dredge

A dragline dredge is very similar to a clamshell dredge. The dragline is a crane-operated device that is normally mounted on a flat-bottomed barge or pontoon. Like the clamshell dredge, it can also be crawler-mounted. The dragline may be used to excavate almost any type of material, just as the clamshell.

The primary difference between the dragline and the clamshell is in the control cable arrangement. The dragline bucket is loaded by being pulled by a drag cable through the material being excavated and toward the crane. By this arrangement, the dragline offers a longer reach than the clamshell (Merritt, 1976). The dragline is illustrated in Figure 8-3. To accommodate the control cable arrangement, the bucket of the dragline is somewhat different than the clamshell.

FIGURE 8-3.
DRAGLINE BUCKET ON TRACK-MOUNTED CRANE



Source: Merritt, 1976

a. Applications/Limitations

Because the dragline employs the same basic equipment as the clamshell dredge, its applications and limitations are very similar to those of the clamshell. There are, however, some slight differences.

Draglines are adaptable to either land-based or barge-mounted operation. They are capable of excavating material at nearly in-situ densities and are easily controlled and maneuvered in small and confined areas, although somewhat less than the clamshell. Draglines are capable of shallow water excavation. Their availability in the United States is relatively good.

Major limitations of draglines are that they cause a great deal of turbidity and sediment resuspension and their production rate is low. Like the clamshell, they are inefficient in terms of losing material from the bucket during the hoisting operation and they require separate disposal vessels and equipment for operation. They are limited to use in shallow streams and rivers with low flow velocity, and are considered ineffective against a free or unadsorbed liquid contaminant (Merritt et al., 1976).

8.1.1.3 Backhoes

Backhoes are normally used for trenching and for other subsurface excavation where it is expedient to keep the excavator at original ground level. Backhoes are mechanically or hydraulically operated in a drag and hoist maneuver. They are usually crawler-mounted, although they can be barge-mounted. The lateral and vertical reach of a backhoe is limited by the length of the boom. Conventional backhoes are capable of digging to a depth of about 40 feet. Extended backhoes are also available and can dig to depth of up to 80 feet.

Because of the limited lateral and vertical reach of backhoes, they are not often used for the removal of contaminated sediments. However, availability of backhoes is excellent. They are also capable of excavating almost any type of material, and are easily controlled and maneuvered in small and confined areas. On the other hand, backhoes share the limitations of other mechanical dredging terchniques, such as low production rate and requirements for separate disposal vessels and equipment.

Backhoes and other conventional excavating equipment are discussed in greater detail in Chapter 7.

8.1.1.4 Bucket Ladder Dredge

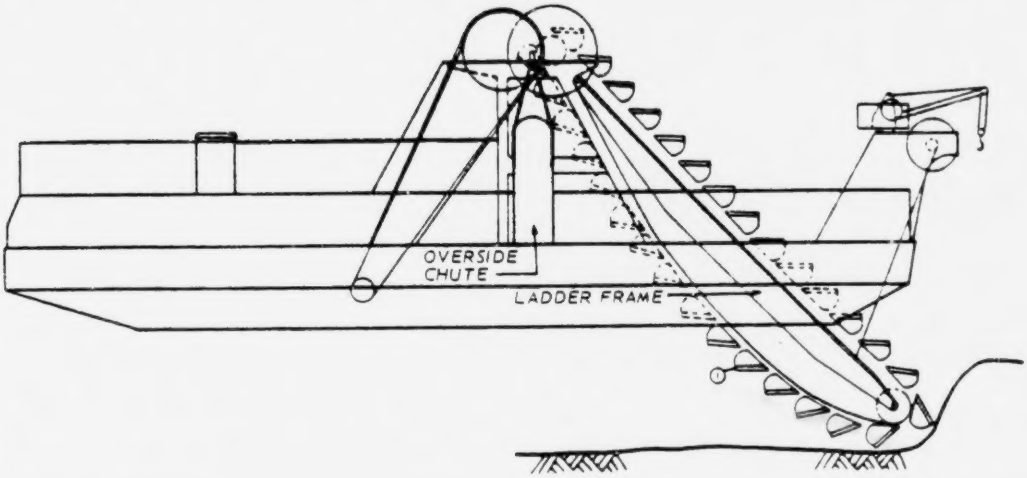
The dredging action of the bucket ladder dredge (Figure 8-4) is provided by an inclined submersible ladder which supports a continuous chain of buckets that rotate around pivots at each end of the ladder. As the buckets rotate around the bottom of the ladder, they scoop up the sediment which is then transported up the ladder and dumped into a storage area as the buckets round the top pivot. Most bucket dredges are mounted on pontoons and are not self-propelled. Only four of these dredges are known to be operating in the United States (Hand et al., 1978).

a. Applications/Limitations

Bucket ladder dredges are most commonly used in mining operations such as sand and gravel production. Production rates are generally higher than for other types of mechanical dredges, and they can handle many different kinds of material. Bucket ladder dredges can also load barges. This is an important factor in cases where dredged material must be transpsorted over long distances to a disposal site. Bucket volumes range from 2.8 to 36 cubic feet. The average dredging depth of the bucket ladder is 60 feet but some can go as deep as 100 feet (Hand et al., 1978).

The major limitations of the bucket dredge is the high degree of turbidity generated by the mechanical agitation of sediments and bucket leakage. Bucket ladder dredges require a great deal of support equipment (for boats and barges), and require a complicated configuration of mooring lines. The relative unavailability in the United States make bucket dredges unlikely candidates for dredging of hazardous materials (Hand et al., 1978).

FIGURE 8-4.
BUCKET LADDER



Source: Hand et al., 1978

8.1.2 Hydraulic Dredging

Hydraulic dredges remove and transport sediment in liquid slurry form. Slurries of 10 to 20 percent solids by wet weight are common in standard hydraulic dredging operations. The slurries may be pumped many thousands of feet through floating or pontoon-supported pipeline to a dredged material treatment/storage area. Hydraulic dredges are usually barge-mounted and carry diesel- or electric-powered centrifugal pumps with discharge pipes ranging in size from 6 to 48 inches in diameter. Unlike mechanical dredges, they can be used in waters with appreciable flow velocity.

The suction end of a hydraulic dredge is mounted on a moveable ladder which may be lowered or raised to a specified dredging depth. Often, the suction end of the dredge is fitted with a cutterhead to assist in cutting.

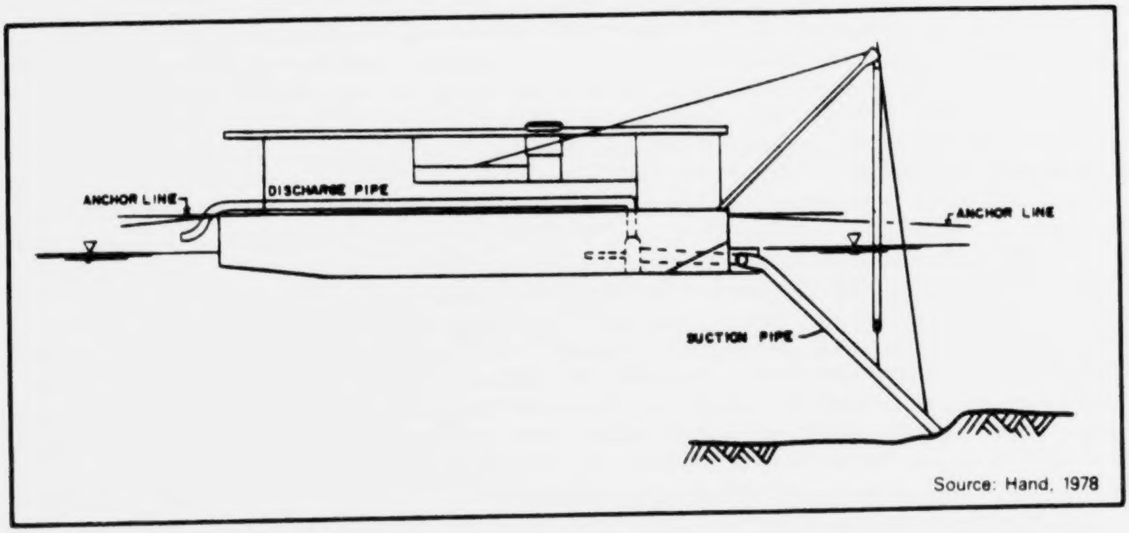
The major disadvantage of hydraulic dredges is a large flow rate associated with pumping at low solids concentrations, resulting in the need for large areas of land to serve as settling/dewatering areas for dredged material. Recently, emphasis in research has been in the development of hydraulic dredges which are capable of removing sediments at near in-situ solids concentrations, minimizing the water content of the pumped slurry, and thereby lowering the land requirements for sediment dewatering. Another area of emphasis in dredge design is in the development of cutterheads which offer low turbidity generation characteristics through the use of shrouds and other auxiliary features. Hydraulic dredges include plain suction, cutterhead, dustpan, and hopper.

8.1.2.1 Plain Suction Dredge

The plain suction dredge is the simplest of all hydraulic dredges. It relies solely on the suction generated by the centrifugal pump to dislodge, capture, and transport the excavated slurry. The dredge head is attached to the end of a ladder and its position is controlled vertically and horizontally by the movement of cables attached to the ladder. Figure 8-5 illustrates a plain suction dredge. During normal operations, material with solids contents of 10 to 15 percent by weight are drawn from the bottom up through the suction line and discharged into a scow or through a pipeline to a nearby disposal site. The production rate of suction dredges depends upon the pump size, pump horsepower, and type of material being dredged. During normal working conditions, dredging is performed at 1,000 to 10,000 cubic yards per hour, depending on the discharge velocity and pipe diameter (Hand et al., 1978).

Plain suction dredges are normally pulled along a straight line fixed by a cable-and-winch arrangement anchored on land or on the bottom of the watercourse. The dredge vessel moves along the line of the cable, and the cable is repositioned to establish a new line as dredging progresses. They have no capability for lateral manipulation beyond the positioning and movement of the dredge vessel. Vertical control of sediment removal is maintained by raising and lowering the suction pipe and dredge head supporting ladder using a cable-and-winch arrangement (Hand et al., 1978).

FIGURE 8-5.
PLAIN SUCTION DREDGE



Source: Hand, 1978

a. Applications/Limitations

Plain suction dredges are effective in the removal of relatively free flowing sediments such as sands, gravels, and unconsolidated material, and can operate without difficulty in deep waters. The most advantageous features of the plain suction dredge are its high production rate and its ability to discharge directly to disposal areas, thus eliminating the need for extra equipment.

Plain suction dredges are ineffective against hard and cohesive materials such as clays and native bottom sediments, as they employ no mechanical dislodging devices. Although plain suction dredges are capable of dredging large volumes of material, this material is 80 to 90 percent water. This demands extensive dewatering and consolidation of the material prior to disposal. Plain suction dredges should not be operated in rough waters. Additionally, the anchoring cables and pipelines often cause obstructions to river traffic. The suction line is subject to blockage or damage by underwater debris. There are about two dozen plain suction dredges operating in the United States (Hand et al., 1978).

8.1.2.2 Cutterhead Dredge

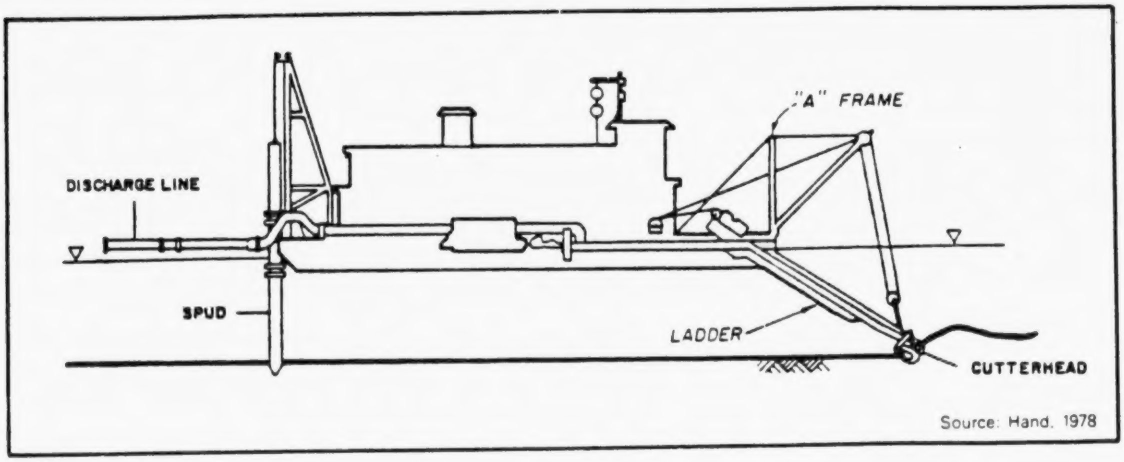
The cutterhead is probably the most efficient and versatile dredge of all. Its configuration is similar to the plain suction dredge, except that it is equipped with a rotating cutter apparatus surrounding the intake end of the suction pipe. This device, known as the cutterhead, rotates to dislodge sediment and allows transport of sediment by suction to the suction pipe. Slurries of 10 to 20 percent solids by weight are typically achieved, depending upon the material being dredged (Hand et al., 1978). Figure 8-6 illustrates a cutterhead dredge.

Cutterhead dredges move in a pattern different from other hydraulic dredges by alternately anchoring on one of two spuds. The anchored spud is used as a pivot and the vessel is drawn along an anchored cable, thus swinging the cutterhead in a short horizontal arc about the spud. Repeated swinging of the cutterhead arcs while alternating anchored spuds results in partially overlapping cuts which form a wide effective cut through the area being dredged (Hand et al., 1978).

a. Applications/Limitations

The cutterhead's efficiency and versatility are the features that place it among the most popular dredges in the world. Cutterhead dredges are the workhorses of the U.S. dredging fleet with close to 300 in use nationwide (Hand et al., 1978).

FIGURE 8-6.
CUTTERHEAD DREDGE



Source: Hand, 1978

The cutterhead can efficiently dig and pump all types of alluvial materials or compacted deposits such as clay or hardpan. The larger and more powerful machines are used to dredge rock-like formations such as coral and the softer type of basalt and limestone without blasting. The cutterhead is capable of constructing level bottoms and finishing slopes efficiently.

A properly designed cutterhead will cut and guide the bottom material toward the suction efficiently, although the cutting action and the turbulence associated with the rotation of the cutterhead resuspend a portion of the bottom material. The ability of the dredge's suction to pick up bottom material determines the amount of cut material that remains on the bottom or is resuspended (Raymond, 1983).

Little experimental work on cutterhead resuspension has been done. Field studies have revealed that suspended solids concentrations are highly variable within 10 feet of the cutter, but may be as high as a few tens of grams per liter. These concentrations decrease exponentially with depth from the cutter to the water surface. Near-bottom suspended solids concentrations may be elevated to levels of a few hundred milligrams per liter at distances of 1,000 feet from the cutter (Raymond, 1983).

8.1.2.3 Dustpan Dredge

The dustpan dredge is a hydraulic suction dredge which features a widely flared dredging head along which are mounted high-pressure water jets. These jets loosen and agitate the sediments which are then captured in the dustpan head as the dredge itself is winched forward into the excavation. Dredge material slurries with solids contents of 10 to 20 percent by weight are common with dustpan dredges (Raymond, 1983). A dustpan dredge is illustrated in Figure 8-7.

a. Applications/Limitations

The dustpan dredge works best in free-flowing granular material. The high pressure jetting action may improve efficiency slightly by loosening cohesive deposits. Production rates for dustpan dredges are high, about the same as for plain suction dredges (Raymond, 1983).

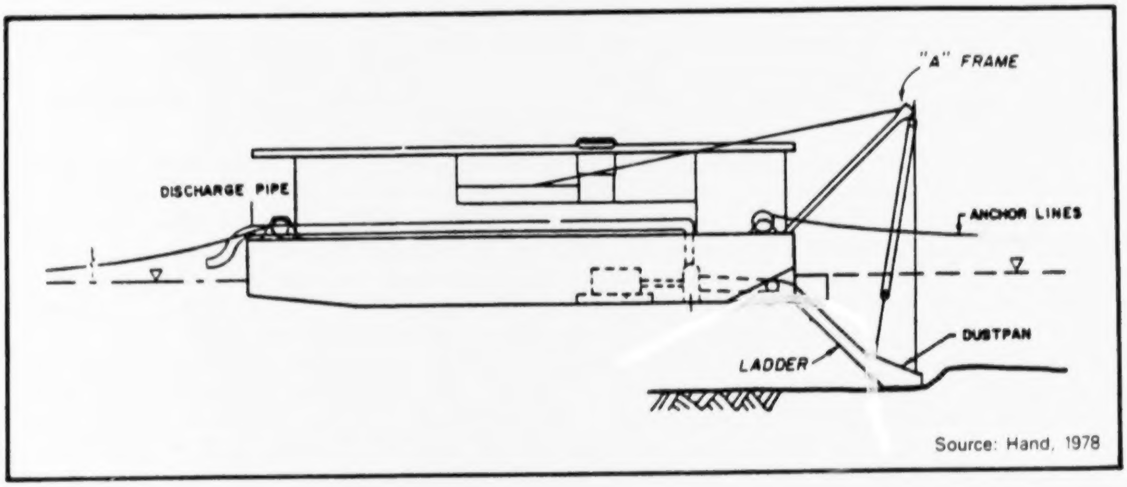
Considerable resuspension of fine-grained sediments may result from the jetting action of the dustpan dredge (Hand et al., 1978). The Norfolk District of the Corps of Engineers conducted a demonstration project in 1982 using a modified dustpan head (without water jets) to dredge the fine-grained sediments of the James River in Virginia. It was hoped that the dustpan head, using suction only, could excavate thin layers of the contaminated clay sediment with less resuspension than a cutterhead. The dustpan head, however, experienced repeated clogging and produced at least as much resuspension as a cutterhead operating in the same material (Raymond, 1983). Availability of the dustpan dredge is limited; there are about one dozen duspan dredges in the United States. All of them are owned by the U.S. Corps of Engineers and are used primarily for channel maintenance in interior waterways.

8.1.2.4 Hopper Dredges

Hopper dredges (Figure 8-8) differ from other hydraulic dredges primarily in the type of vessel used and the methods of attachment and operation of the dredge head. Hopper dredge vessels are normally large, self-propelled, sea-going vessels, rather than barges. They are positioned and moved by the propeller/rudder navigating equipment of the host vessel.

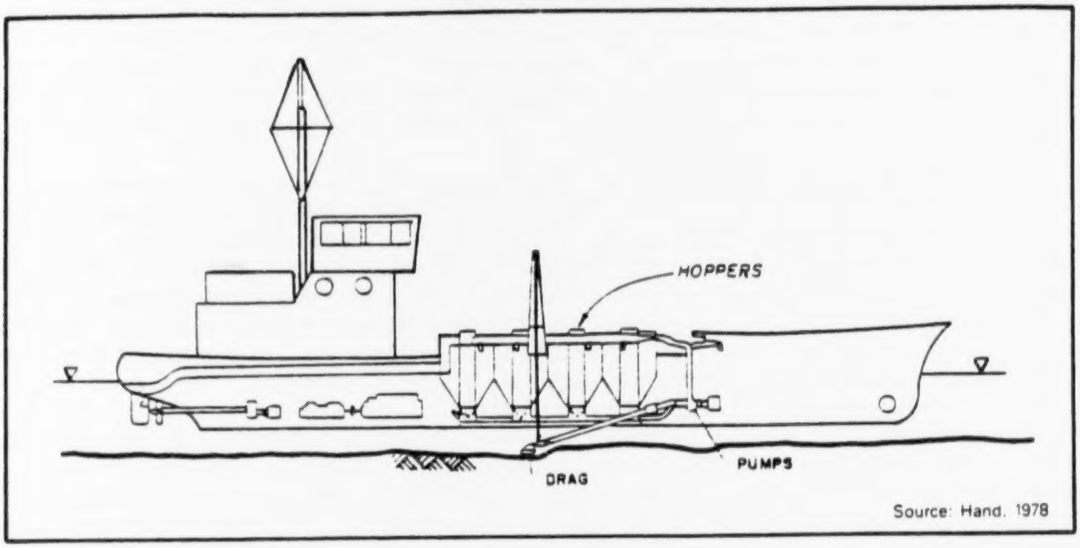
The suction pipes are hinged on either side of the vessel and extend downward toward the stern of the vessel. The dredge heads, attached at the end of the suction pipes, drag along the bed of the area being dredged as the vessel moves forward; the head is sometimes called a "trailing" head for this reason. Dredging is accomplished by the vessel making progressive passes over the project area. Dredged material is transported up the suction pipe and is discharged for storage into the hopper of the vessel. Coarse-grained material settles to the bottom of the hopper and water and fine-grained sediment is normally allowed to overflow the hopper into the water course. (Overflow would usually be unacceptable in the removal of contaminated sediments.) Once fully loaded, the vessel moves to an unloading area where emptying of the hopper is accomplished by opening bottom doors or by pumping the contents to a treatment on disposal area. There are 15 ocean-going, trailing suction hopper dredges operated by the Corps of Engineers as well as several privately-owned vessels (Barnard, 1978, and Raymond, 1983).

FIGURE 8-7.
DUSTPAN DREDGE



Source: Hand, 1978

FIGURE 8-8.
HOPPER DREDGE



Source: Hand, 1978

a. Applications/Limitations

Hopper dredges are intended for maintenance dredging for deep, rough-water shipping channels, and are normally most efficient in excavating loose, uncohesive materials. They can reach sediment up to 62 feet deep and, when fully loaded, draft in the range of 13 to 24 feet. Therefore, hopper dredges are limited to dredging of deep harbors and off-shore water courses. A hopper dredge is capable of operating in rough, open waters; in relatively high currents; in and around marine shipping traffic; and in adverse weather conditions (Hand et al., 1978).

Field data confirm that the suspended solids levels generated by a hopper dredge operation are primarily caused by hopper overflow in the near-surface water and draghead resuspension in near-bottom water. Suspended solids concentrations may be as high as several tens of grams per liter near the discharge port and as high as a few grams per liter near the draghead. Suspended sediment levels in the near-surface plume may exceed background levels even at distances in excess of 3,600 feet (Barnard, 1978 and Raymond, 1983).

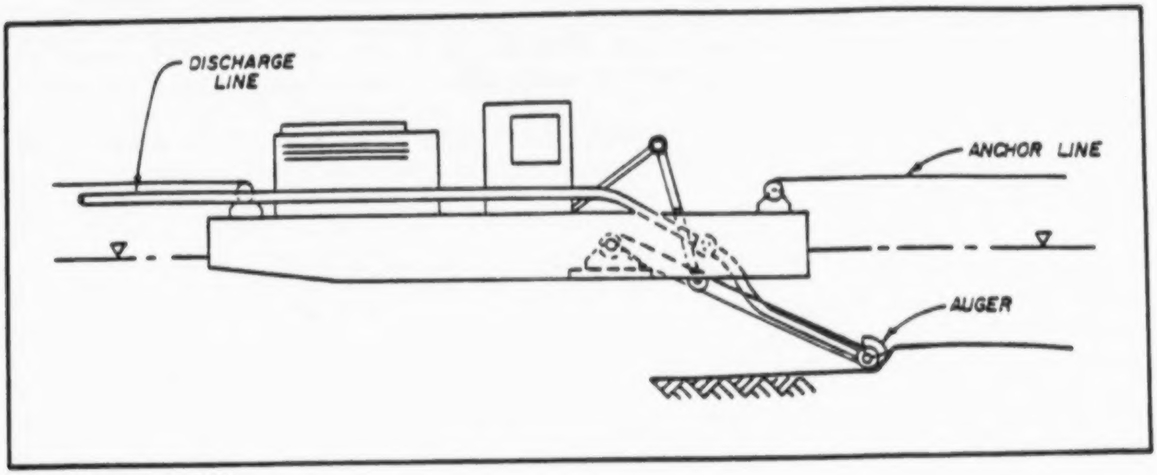
8.1.2.5 Portable Hydraulic Dredges

Portable hydraulic dredges are defined as dredge vessels which can be moved easily from one job site to the next over existing roadways without major dismantling. The U.S. Army Corps of Engineers Waterways Experiment Station has prepared a "Survey of Portable Hydraulic Dredges" (Clark, 1983), which is a compilation and assessment of models of portable dredges available in the United States. Conventional cutterheads, horizontal cutters, bucket wheel, chain cutters, vertical cutters, and dustpans are available on portable dredges, and have dredging depth capabilities ranging from 10 to 50 feet. Among the manufacturers of portable dredges are: Mud Cat; Vaughn-Maitlen Industries (VMI); Ellicott Machine Corporation International; Eagle Iron Works; W&S Development, Inc.; American Marine & Machinery Company, Inc., (AMMCO); Quality Industries, Inc.; Dredge Masters International (DMI); Delta Dredge and Pump Corporation; Mini Dredge, Ltd.; General Conveyors Limited (GENFLO); Kenner Marine and Machinery, Inc.; and Waterless Dredging Company.

An example of the most commonly used portable dredge, a Mud Cat, is shown in Figure 8-9. It is available in several models with depth capabilities up to 15 feet.

The Mud Cat is pontoon-mounted and features a horizontally mounted, auger-like cutting device that feeds the sediment to the suction intake of a diesel-driven centrifugal pump. The auger is mounted along the base of a bulldozer-type blade. The whole arrangement with suction pipe attached is controlled by a hydraulic boom. The dredge is moved along on an anchored cable during each traverse of the excavation, and the dredge material is discharged ashore through a float-supported pipeline. The width of the cut is approximately 8 feet, and applications to date have included dredging of small

FIGURE 8-9.
PORTABLE HYDRAULIC DREDGE



Source: Hand et al., 1978

reservoirs and streams. There are several hundred Mud Cat dredges owned and available for lease from the National Car Rental Agency.

Vaughn Company, Inc., and Mud Cat have developed the Vaughn Lagoon Pumper, a highly portable electric pumping unit mounted on a platform between two polyethylene floats. The unit was proved an efficient, inexpensive means of removing settled solids from industrial and organic ponds with less dilution and lower waste hauling and disposing costs. (Mud Cat, 1983). The unit is 7-1/2 feet wide, 18 to 24 feet long, and weighs 4,500 pounds. It features a chopper pump that is capable of dredging sludges, weeds, peat, silt, and organic material. Like the Mud Cat, the Lagoon Pumper is driven by a winch-and-cable arrangement and can be equipped for either on-board operation or on-shore remote control (Vaughn Co., Inc., 1985).

a. Applications/Limitations

Portable dredges are particularly applicable to contaminated sediment remediation projects in relatively isolated (in terms of navigation) water bodies such as lakes and inland rivers. Portable dredges also characteristically have low depths of vessel draft (less than 5 feet and many less than 2 feet), allowing them to be used in shallow-water applications (Vaughn Co., Inc., 1985).

A series of tests on the Mud Cat sponsored by EPA in the 1970s, showed that resuspension of the sediment was low and the resuspension plume imparted to the surrounding water during dredging was confined to within 20 feet of the dredge (Hittman, 1974). The Mud Cat was also found to be very efficient in removing simulated hazardous material (powder, fine glass beads, filings, and coal), removing 99.3 percent from the sediments of a test pond (Hittman, 1976).

The Vaughn Lagoon Pumper can operate in 1.5 feet of water and can remove materials to depths of 11 feet. Pumping capacity is up to 100 gallons per minute and 150 feet of discharge head (Mud Cat, 1983). The scale of the Lagoon Pumper offers a compromise between hand-held, abovewater vacuum dredges and the conventional portable hydraulic dredges.

The primary limitations of portable dredges is their size. Small size restricts portable dredges to low production rates, which limit them to small-scale operations.

8.1.2.6 Hand-Held Hydraulic Dredges

A variety of hand-held dredges have been adapted from equipment designed for other applications. Hand-held dredges remove a mixture of sediment and water in a slurry. They may be used either abovewater or underwater.

Underwater hand-held dredges are normally operated by divers. The equipment employed can range from a hose-and-collector arrangement to skid-mounted, high-production machines used for mineral recovery and maintenance at off-shore platforms. Abovewater, hand-held dredges are normally used by operators wading into shallow waterways or from small water craft. Cleanup of PCB-contaminated sediments in the South Branch of the Shiawassee River in Michigan was accomplished largely by vacuum dredging with hand-held hoses controlled by wading operators. Vacuum trucks were used as the source of vacuum and for temporary storage and transport of dredged slurry. Vacuum hoses extended from the shore-based trucks to the areas being dredged. Also, a relatively simple vacuum unit employing diaphragm sludge pumps was used by the EPA Region X Inland Regional Response Team to remove PCB-contaminated sediments from the Duwamish Waterway in Washington (Willmann, Blazevich and Snyder, 1976).

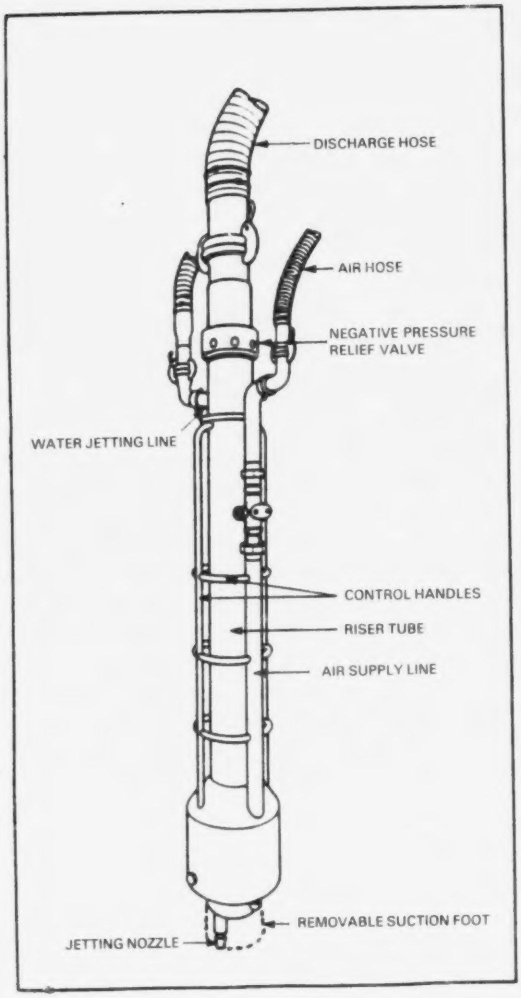
Dredging equipment has also been specially designed for operation by divers although it has not been used in hazardous waste site remediation. Alluvial Mining and Shaft Sinking Co., Ltd., a British dredging engineering and manufacturing firm, manufactures diver-operated airlift dredges and a vibrating hydraulic "clay spade" for removing cohesive sediments which are both shown in Figure 8-10 (Hand et al., 1978).

a. Applications/Limitations

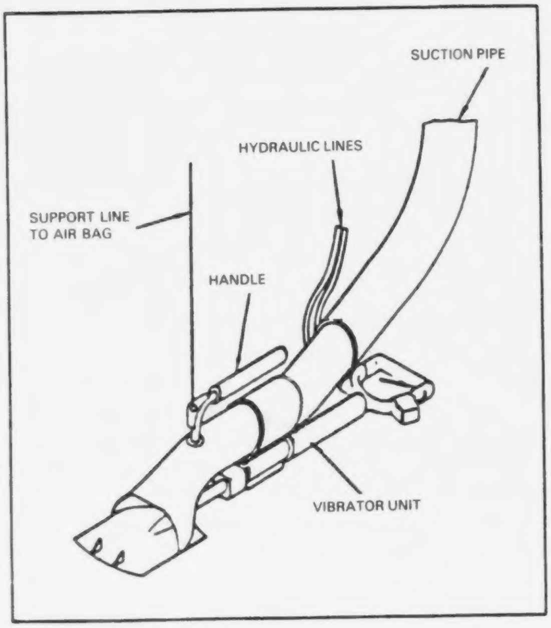
The major applications of hand-held hydraulic dredges is for small-volume projects in relatively calm waters where precision dredging is important. Hand-held dredges can be quickly designed and assembled from off-the-shelf equipment (Hand et al., 1978).

FIGURE 8-10.
DIVER-OPERATED HAND-HELD DREDGES

AIRLIFT DREDGE



CLAY SPADE DREDGE



Source: Alluvial Mining, 1984

Abovewater units are limited to shallow or dewatered water bodies. Resuspension of contaminted sediments is aggravated by wading operators or divers disturbing bottom sediments (Hand et al., 1978).

8.1.2.7 Waterless Dredge

The Waterless dredge, developed by Waterless Dredge Company, was designed for removal of sludges from lagoons at a minimum water content. Special components of the dredge include a shrouded "roll-over" cutterhead and a submerged dredge pump. The cutterhead is open and fills with sediment (or sludge). The sediment displaces water in the cutterhead and blocks entry of water into the dredge pump inlet, and only sediment is pumped. At the limit of the swing to the left, when the bow of the dredge begins to swing to the right, the cutterhead rolls down and over 180 degrees so that the face of the cutterhead is open to receive sediment and repeat the process. The dredge reaches depths up to 16 feet, and the Waterless Dredge Company claims that pumped slurry contains less than 10 percent excess water, by volume (World Dredging and Marine Construction, 1980).

a. Application/Limitation

The Waterless Dredge was used to remove lead-contaminated sediments from the Mill River in Connecticut. Resuspension and resettling of contaminated sediment was apparently of sufficient magnitude to warrant a second dredging of recontaminated areas (York Wastewater Consultants, 1983).

8.1.3 Pneumatic Dredging

Pneumatic dredges are treated as a distinct category from hydraulic dredges only because of their novelty in this country. Originally developed in Italy under the trade name Pneuma, these systems feature a pump that operates on compressed air and hydrostatic pressure to draw sediments to the collection head and through the transport piping. Otherwise, they are no different than hydraulic dredges. There are several different so-called pneumatic dredges, including the airlift, the pneuma, and the oozer. Pneumatic dredges can be operated in shallow or deep water with no theoretical maximum depth, and can be relatively easily dismantled and transported by truck or air. Pneumatic dredges may be able to yield denser slurries than conventional hydraulic dredges with lower levels of turbidity and resuspension of solids (Hand et al., 1978).

One of the major limitations of the pneumatic dredges is that they are capable of only modest production rates (up to 390 cubic yards/hour). Another problem is that cables and pipelines present temporary obstructions in navigable water channels. Pneumatic systems are not in widespread use in the United States, and may not be as readily available as other types.

8.1.3.1 Airlift

Airlift dredges use compressed air to dislodge and transport sediments. Compressed air is introduced into the bottom of an open vertical pipe, usually controlled and supported by a barge-mounted crane. As the air is released, it expands and rises, creating upward currents which carry both water and sediment up the pipe. The applied air pressure must be sufficient to overcome the hydrostatic pressure at operating depths. Higher air pressures and flow rates result in higher transport capacity. Air can also be introduced through a special transport head which can be vibrated or rotated to further dislodge more cohesive sediments. Slurries of 1:3 solid/liquid ratio can typically be achieved with airlift dredges (d'Angremond, 1978).

Airlift dredges are crane-supported and can be mounted on barges, and sea-going vessels, as well as dockside. Lateral control is achieved by swinging the boom of the crane in a manner similar to mechanical dredging. Vertical control is achieved by raising and lowering the open end of the vertical transport pipe and by varying the pressure of the air released at the end of the pipe (Hand et al., 1978).

a. Applications/Limitations

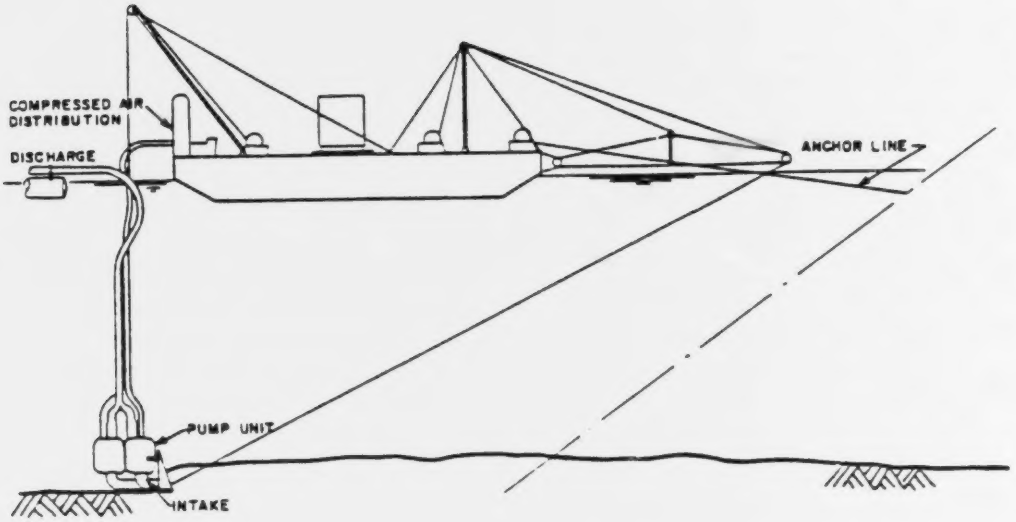
The primary advantage of the airlift dredge is that it provides continuous transport of material, thus maximizing production rate. The primary limitation is that sufficient depth must be available in order to build up enough air pressure for operation. The minimum dredging depth for economical operation is about 20 to 30 feet (Hand et al., 1978).

8.1.3.2 Pneuma Dredge

The Pneuma dredge (Figure 8-11) consists of a pump which is lowered by a crane into the sediments being dredged. The pump is driven by compressed air and operates by positive displacement. The body of the pump contains three cylindrical vessels, each with an intake opening on the bottom and an air port and a discharge outlet on top. The air ports can be opened to the atmosphere through air hoses and valves. The three cylinders operate in parallel, each one-third cycle ahead and behind the other two cylinders, and controlled by an air distributor located on the control vessel (Richardson, 1982).

Pneuma dredges are normally suspended from a crane cable and pulled ahead into the sediments being dredged by a second cable. The dredge head is essentially fixed relative to the vessel so that lateral manipulation of the dredge is limited to the positioning and movement of the vessel.

FIGURE 8-11.
PNEUMA DREDGE



Source: Hand. 1978

a. Applications/Limitations

Though Pneuma dredges are most applicable to loosely consolidated sediments, the intake openings can be fitted with shovel attachments to aid in penetration of sediments. Extremely deep applications are possible, limited by vertical and lateral controllability and air pressure requirements. They are capable of delivering a slurry of high solids content with minimal turbidity generation. Tests showed that the pneuma pump was able to dredge at almost in-situ density in a loosely compacted, silty clay typical of many estuarine sediments while generating a low level of turbidity (Richardson, 1982). Though not small, Pneuma dredges are relatively easily dismantled and transported by truck or air. The operation of the Pneuma is partially dependent upon hydrostatic pressure, and this may limit its effectiveness in shallow water (Richardson, 1982). Another limitation is that Pneuma dredges have limited availability in the United States.

As a pure pumping device, the Pneuma pump has a very low power efficiency compared with a conventional centrifugal pump. Efficiencies in pumping sand or silty clay are less than 20 percent and often less than 10 percent (Richardson, 1982).

Pneuma dredges are also only capable of modest production rates (up to 390 cubic yards per hour), and the cables and pipelines present temporary obstructions in navigable water channels (Richardson, 1982).

8.1.3.3 Oozer Dredge

The Oozer dredge, developed in Japan, consists of a pump similar in concept to the Pneuma. It uses negative (vacuum) pressure in the filling chambers and atmospheric pressure when dredging in the shallow waterways. The pump is usually mounted at the end of a ladder. The pump body consists of two cylinders to which a vaccum is applied to increase the differential pressure and flow between the sediment and the cylinders. Sediment thickness detectors, underwater television cameras, and a turbidimeter are attached near the suction mouth for monitoring. Suspended oil can be collected by an attached hood, and cutters can be attached for dislodging hard soils (Barnard, 1978 and Toyo Construction, Co., Ltd., undated).

Oozer dredges are normally pulled along a straight line fixed by a cable-and-winch arrangement anchored on land or on the bottom of the watercourse. The dredge vessel moves along the line of the cable, and the cable is repositioned to establish a new line as dredging progresses.

a. Applications/Limitations

The Oozer dredge is capable of operating at depths up to 60 feet. Toyo Construction Co., Ltd., of Tokyo claims slurries of 30 to 70 percent solids content can be achieved without significantly increasing turbidity or causing resuspension (Barnard, 1978 and Toyo Construction Co., Ltd., undated).

Moderate production rates and obstruction of waterway traffic are limiting factors for Oozer dredges. In addition, they have no capability of lateral manipulation beyond the positioning and movement of the dredge vessel. Vertical control of sediment removal is maintained by raising and lowering the suction pipe and dredge head supporting ladder using a cable-and-winch arrangement. The availability of Oozer dredges is limited in the United States.

8.1.4 Implementation of Dredging Operations

This section describes predredging activities, dredge vessel control, and turbidity control.

8.1.4.1 Predredging Operations

Predredging operations consist of a variety of tasks that are conducted prior to and in preparation for the actual dredging project. Predredging operations may include site surveys, site operation plans, equipment mobilization and demobilization, stream diversion, cofferdam construction, weed harvesting, and bottom snagging.

a. Stream Diversion

In some contaminated sediment areas, complete hydraulic isolation of sediments may be desirable so that dewatering followed by dry excavation may be implemented, or so that hydraulic dredging may be conducted in a contained environment. To accomplish stream diversion, all or part of the flow is diverted by cutting off a section of the stream, diverting flow to a pipe or excavated channel, and allowing the flow to re-enter the stream channel at a point further downstream. Stream diversion may be accomplished by placement of cofferdams (described in the next section), and may be temporary or permanent. Generally, permanent diversion is practical only for very small streams.

The Environmental Emergency Response Unit, EPA, Edison, NJ, has developed a mobile stream diversion unit for such operations. The unit consists of submersible pumps, booster pumps, generators, a crane, and an aluminum irrigation pipe with ancillary fittings. The system is capable of pumping 5600 gallons per minute over a distance of 1000 feet over level terrain. Additional capacity is available with supplemental piping and pumps.

b. Cofferdams

Cofferdams can be built around a contaminated area in a waterbody to isolate that area from stream flow. The area can then be dredged, dewatered, and excavated, or capped with low permeability material. Cofferdams are most easily constructed for flow containment of shallow ports, streams, and rivers, or waters with low flow velocities. Where flow velocity exceeds 2 feet per second, cofferdam construction is not recommended because of the difficulty of driving sheet piling under these conditions. Cofferdam construction is feasible for some relatively wide and deep rivers (up to about 10 feet), providing that the velocity of flow is not excessive.

Cofferdams may be constructed of many materials, such as soil, sheet piling, earth-filled sheet pile cells, and sand bags (for short duration structures). Sheet pile cofferdams are generally constructed of black steel sheeting from 5 to 12 gauge in thickness, and from 4 to 40 feet in length.

For additional corrosion protection, galvanized or aluminized coatings are available. Cofferdams may be either single walled or cellular, earth-filled sections. Single wall cofferdams may be strengthened by an earth fill on both sides. They are most suited to shallow waters. For depths greater than 5 feet, cellular cofferdams (circular sheet pile cells filled with earth) are recommended (Linsley and Franzini, 1979).

Sheet pile cofferdams may be installed in pairs across streams to temporarily isolate areas of contaminated sediment deposition, and allow access for dewatering and excavation. This operation may be required for streams or riverbeds in which contaminated sediments have been deposited completely across the channel cross-section. Such construction requires that the entire streamflow be temporarily diverted through the excavation of a bypass channel, and installation of corrogated metal piping of sufficient diameter to handle stream flow. Such an arrangement is illustrated in Figure 8-12. Alternatively, a single curved or rectangular cofferdam may be constructed to isolate an area along one bank of a stream or river. This method only partially restricts natural flow and does not necessitate construction of a temporary diversion (bypass) channel to convey entire flow around the area of excavation. The single cofferdam design is illustrated in Figure 8-13.

For small narrow streams, sheet piling may be driven by hand with light equipment such as a hand maul or a light pneumatic hammer. For wider, deeper streams where longer sheeting is required and access may be difficult, heavy driving equipment such as a drop hammer or a pneumatic or steam pile driver is necessary. Preassembled (interlocked) sections of sheeting are positioned and driven with the use of a crane for wide, deep rivers. The crane may be operated from a barge. A preconstruction geologic site investigation may be necessary to ensure that bedrock or impervious strata will not interfere with the pile-driving operation.

The length of sheet piling required for cofferdam construction depends on the stream depth, velocity of flow, and nature of the soil into which the sheeting is driven. In general, the ratio of exposed length of sheeting to driven length (unexposed, anchored into soil) should be about 1:1, with 1 to 3 feet of freeboard above the water surface. For example, to construct a cofferdam on a 5-foot deep river requires sheeting approximately 12 feet long: 5 feet driven, 5 feet exposed to flow, and 2 feet freeboard. A greater length may be required if a layer of soft, muddy, or unconsolidated sediments overlies the stable soil stratum into which the sheeting must be driven.

For excavation of contaminated sediments deposited along only one side of the channel, a single curved or jointed cofferdam can be installed to isolate the construction area from streamflow. Such an installation will partially restrict natural flow, creating an increased water level and higher velocity of flow within the restricted area of the channel. To prevent bank overflow and excessive erosion resulting from this restricted flow, it is recommended that a sheet pile containment wall be driven along the opposite bank in the area of restricted flow. Both the cofferdam and sheet pile reinforcement wall

FIGURE 8-12.
STREAMFLOW DIVERSION FOR SEDIMENT EXCAVATION USING TWO COFFERDAMS AND DIVERSION CHANNEL

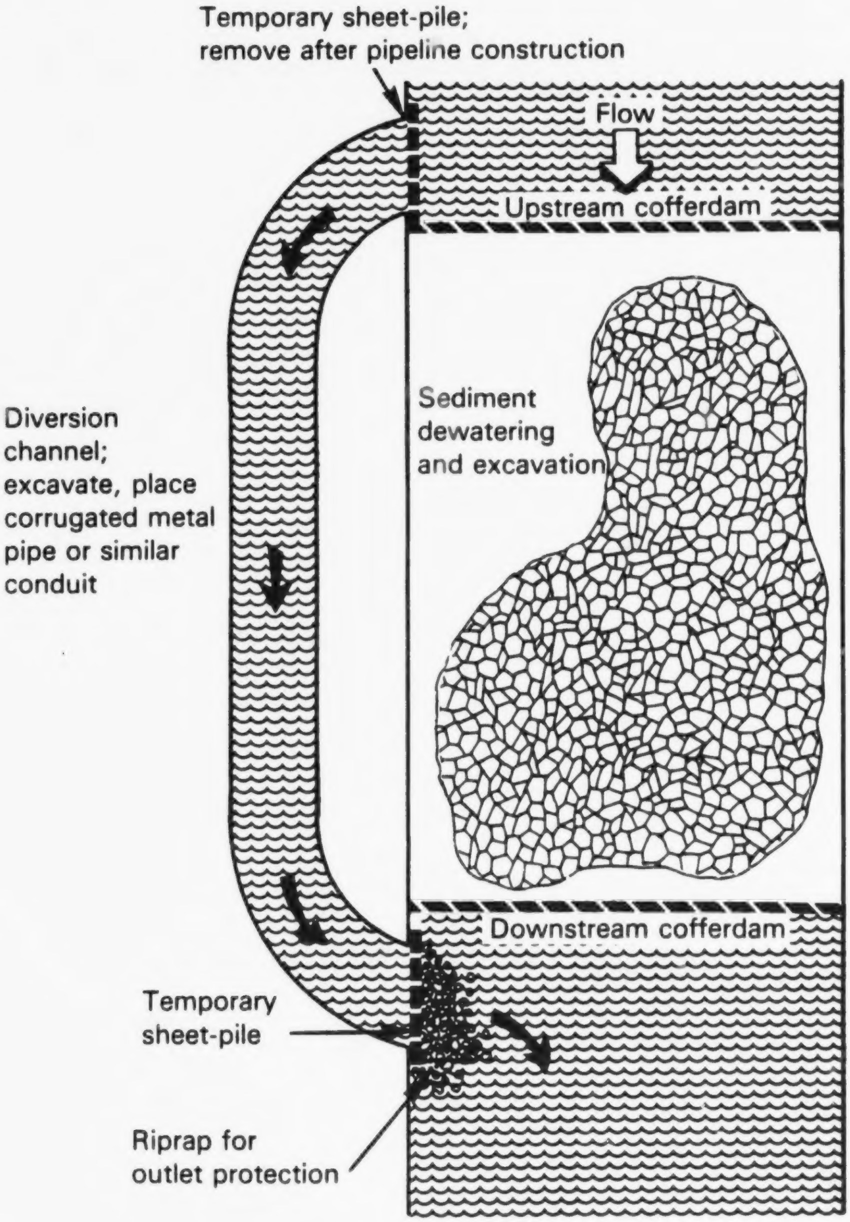
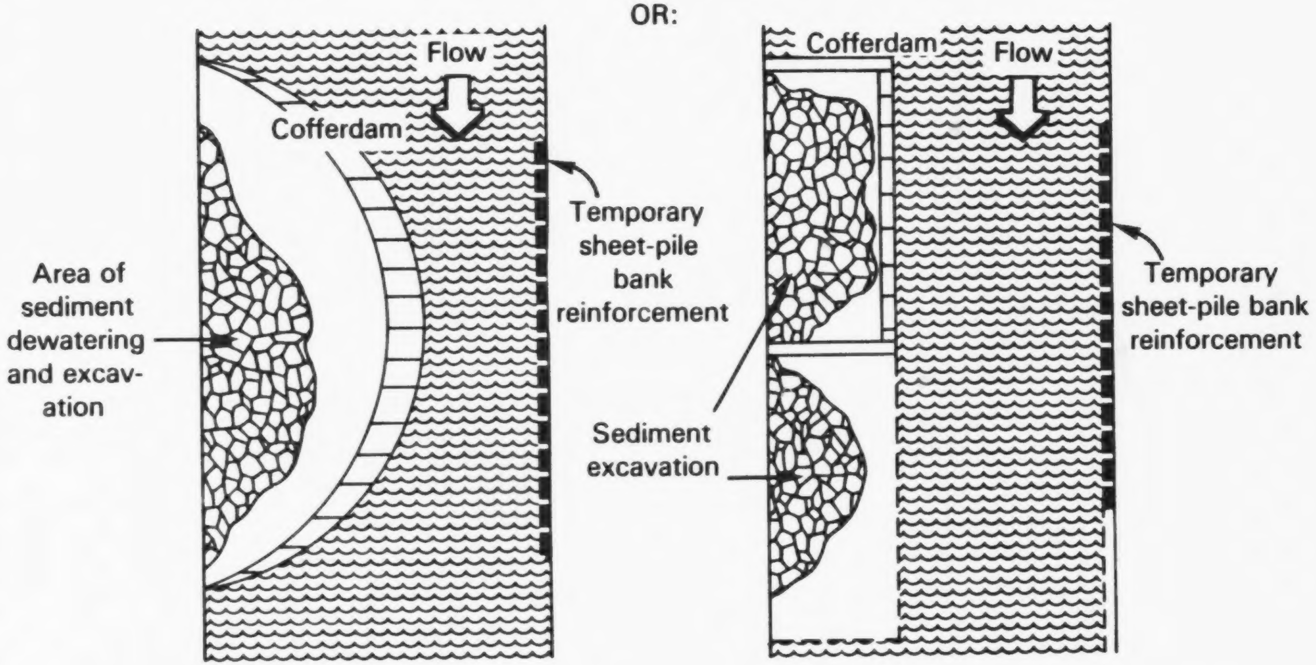

Temporary sheet-pile; remove after pipeline construction
Flow
Upstream cofferdam
Diversion channel; excavate, place corrugated metal pipe or similar conduit
Sediment dewatering and excavation
Downstream cofferdam
Temporary sheet-pile
Riprap for outlet protection

FIGURE 8-13.
STREAMFLOW DIVERSION FOR SEDIMENT EXCAVATION USING SINGLE COFFERDAM


OR:
Flow
Cofferdam
Temporary sheet-pile bank reinforcement
Area of sediment dewatering and excavation
Cofferdam
Flow
Temporary sheet-pile bank reinforcement
Sediment excavation

Source: Barnard, 1978

can be pulled when sediment excavation has been completed, and re-installed further downstream if additional sediment removal is required.

Areas enclosed by cofferdams may require dewatering if infiltration leaks occur through joints in sections of sheet piling, or if excessive precipitation occurs during excavation activities. Dewatering can be accomplished with single-stage centrifugal pumps, which are available in sizes that can pump up to 5,000 gallons per minute (Richardson Engineering Services, 1980). Natural drainage and evaporative drying may be sufficient to dewater small areas, but this may require too much time. Streambed sediments isolated by cofferdams must be sufficiently dewatered before excavation of the contaminated sediments can be performed efficiently.

c. Snagging

Snagging involves the removal of stumps, logs, rubbish, rock debris, and any other objects that might obstruct or damage dredging equipment. Snagging operations are usually carried out with a crane and clamshell or orange-peel grapple. The crane may be barge- or crawler-mounted, depending on the accessibility to the dredge area. In some cases, infestations of aquatic weeds may obstruct dredging and cause delays because of reduced cutterhead mobility and dredge pump clogging. Mud Cat has developed an aquatic weed harvester which can be used to remove aquatic weeds that would obstruct the dredging. Remote sensing and diver survey operations may be required as a prerequisite to locate submerged objects for snagging.

Diver assistance has been required in many snagging operations to secure the grapple to debris in deep water. Also, many activities relating to damage assessment, containment/cleanup activities, environmental assessment, and research studies require that divers enter contaminated environments. Experiences in these environments have often resulted in injuries (primarily chemical burns) to divers and/or surface support personnel. Little information is available on the effects of low level or chronic exposure to these chemicals (McLellan, 1982).

8.1.4.2 Dredge Vessel Control

Precision control is needed to ensure accurate and efficient removal of sediments and minimization of resuspension and transport of contaminants. Types of controls discussed in this section are dredge vessel positioning, monitoring, and control of dredge cut.

The positioning of floating dredge vessels can be considered as gross control over the lateral extent of sediment removal. Positioning and movement of the dredge is generally accomplished using one or more of the following methods:

- Spud anchoring
- Cable-and-winch movement
- Self propulsion (e.g., propellers)
- Towing.

Lateral and vertical control of the dredging of submerged sediment is necessary to ensure that sediment, within identified limits of contamination, is removed, and to minimize over-dredging and resultant costs associated with handling, treatment, and/or disposal of excess material. Precision in dredging is largely dependent upon: (1) the extent to which the dredging equipment can be manipulated (control), and (2) the extent to which the dredge operator is aware of the actual cut achieved (monitoring). The necessary precision required for a given sediment removal situation depends upon the types of contaminants, the lateral and vertical extent of contamination, and the presence of obstacles such as boulders or marine structures. The degree of precision obtainable varies with the dredge equipment. Table 8-1, at the end of this section, summarizes the precision attainable for various types of dredges.

Operators of dredging equipment must receive "feedback" information in some form in order to judge whether the intended extent of sediment removal has been achieved. Visual observations may suffice in some shallow water situations.

However, in many applications, the use of electronic monitoring and control equipment can better provide the operator with information and operating flexibility that is needed in removing contaminated sediments.

Various instruments are capable of providing control or information aiding manual control for the following functions:

- Dredge vessel location and orientation
- Dredge head location and depth
- Depth of cut
- Locations of obstacles
- Slope dredging.

A variety of vessel location-determining systems are available and in use in dredging applications. Most use radio signals or laser beams to determine the vessel's position by triangulation from two or more on-shore reference points, although a laser system which requires only one on-shore station is available (Wentzell, 1983). With the assistance of a minicomputer, one system is capable of determining the coordinates of the vessel's position, within 3 feet, in one second (O'Donnell, 1980). Course deviation meters, which indicate the magnitude and direction of the vessel's deviation from a predetermined course, are also in use. Another system displays the vessel's lateral orientation relative to the heading of a predetermined course, and the distance remaining to the end of a predetermined track to be dredged (Jepsky, 1981).

The depth of the dredge head when it is attached to a ladder (cutterhead or dustpan, for example) is determined through the use of an inclinometer. Mini-computer survey programs are available which provide predredging and postdredging profiles and computations of area and volume dredged, with sounding signals used as direct input.

Precision dredging can be facilitated by marking limits of dredging and obstacles with buoys or buoyed lines. Land-based lasers have been used to mark lateral locations of submerged and buried pipelines. Laser beams are not subject to damage by vessels, are not obstacles to vessel mobility, and are more readily seen at night and during periods of fog (World Dredging, 1983).

8.1.4.3 Sediment Turbidity Control

Removal of contaminated sediments in open water often generates turbidity caused by resuspension of fine-grained particles. Control of turbidity can be accomplished through modification to dredging equipment and the use of curtain barriers. In addition, there are certain procedures that all sediment dredging operations may follow to minimize stream bed agitation and control turbidity. Low turbidity dredging equipment is discussed in previous sections; barriers are discussed in this section.

When dredging in areas of strong currents and natural turbulence, the dredging operation should proceed upstream, into the current, because any turbidity generated must pass around and under the dredge. This will increase the tendency of any suspended material to flocculate and settle. Downstream dredging will allow turbid water to spread ahead of the dredge vessel uninterrupted (Huston, 1976). The effect of controlling turbidity through upstream dredging is greatest when operating in shallow flows.

Another consideration for turbidity reduction is the timing of dredging operations. If dredging is to be done for contaminated sediments in aqueous environments, projects should be scheduled for periods of low flow and dry, calm weather whenever possible. Natural stream turbidity and current turbulence will be minimal at such times and will not contribute to dredge-generated turbidity.

When preparing dredging contracts for contaminated sediment removal where turbidity control is essential, contract provisions should specify the use of special low-turbidity dredge vessels or auxiliary equipment and techniques designed to minimize turbidity generation (Huston, 1976). The bidder should be made to specify minimum sediment removal volumes and maximum allowable turbidity levels in the downstream environment to ensure an effective dredging operation.

Silt Curtains--Silt curtains are low permeability floating barriers that extend vertically from the surface of the water to a specified depth and are used to control near surface turbidity in the vicinity of small dredging and capping operations. A silt curtain must be designed to meet specific site conditions including water depth and changes in water depth due to tides, type of bottom sediment, and current velocity.

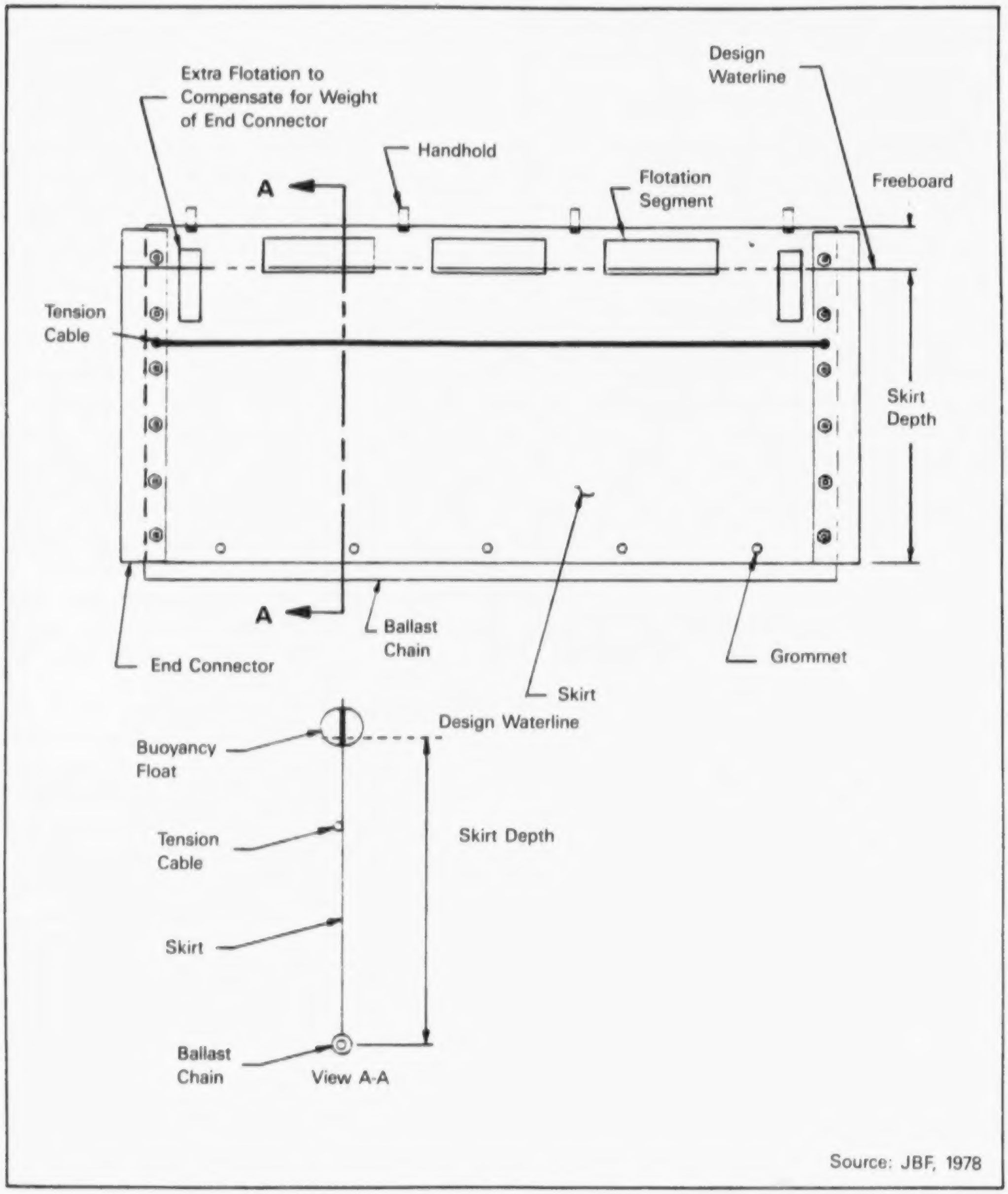
Silt curtains are best suited to use in quiescent waters and in conjunction with dredges that do not demand frequent curtain movement. Under these conditions, turbidity levels outside a curtain may be reduced by 80 to 90 percent (JBF, 1978).

Silt curtains are not recommended for use in open oceans, in currents that exceed 1 knot, in areas frequently exposed to high tides and large waves, or around hopper or cutterhead dredges where frequent curtain movement is necessary. Tides and wave actions cause the curtain to flair, thereby reducing its effective depth. In areas characterized by tidal currents, the use of this method may actually result in higher turbidity levels outside the curtain than inside because of the sweeping motion of the curtain, which may cause resuspension (JBF, 1978).

The components of a silt curtain are illustrated in Figure 8-14. The flexible skirt is generally constructed from polyester reinforced PVC or nylon reinforced PVC. Skirts made of KEVLAR/polyester blend are also available (Slickbar, Inc., 1983). The skirt is maintained by a ballast chain along the bottom. A tension cable is usually built into the upper part of the curtain to absorb stress imposed by currents. End connectors are available to allow two sections of curtain to be attached or disengaged easily. Anchored lines are used to hold the curtain in the deployed configuration (JBF, 1978).

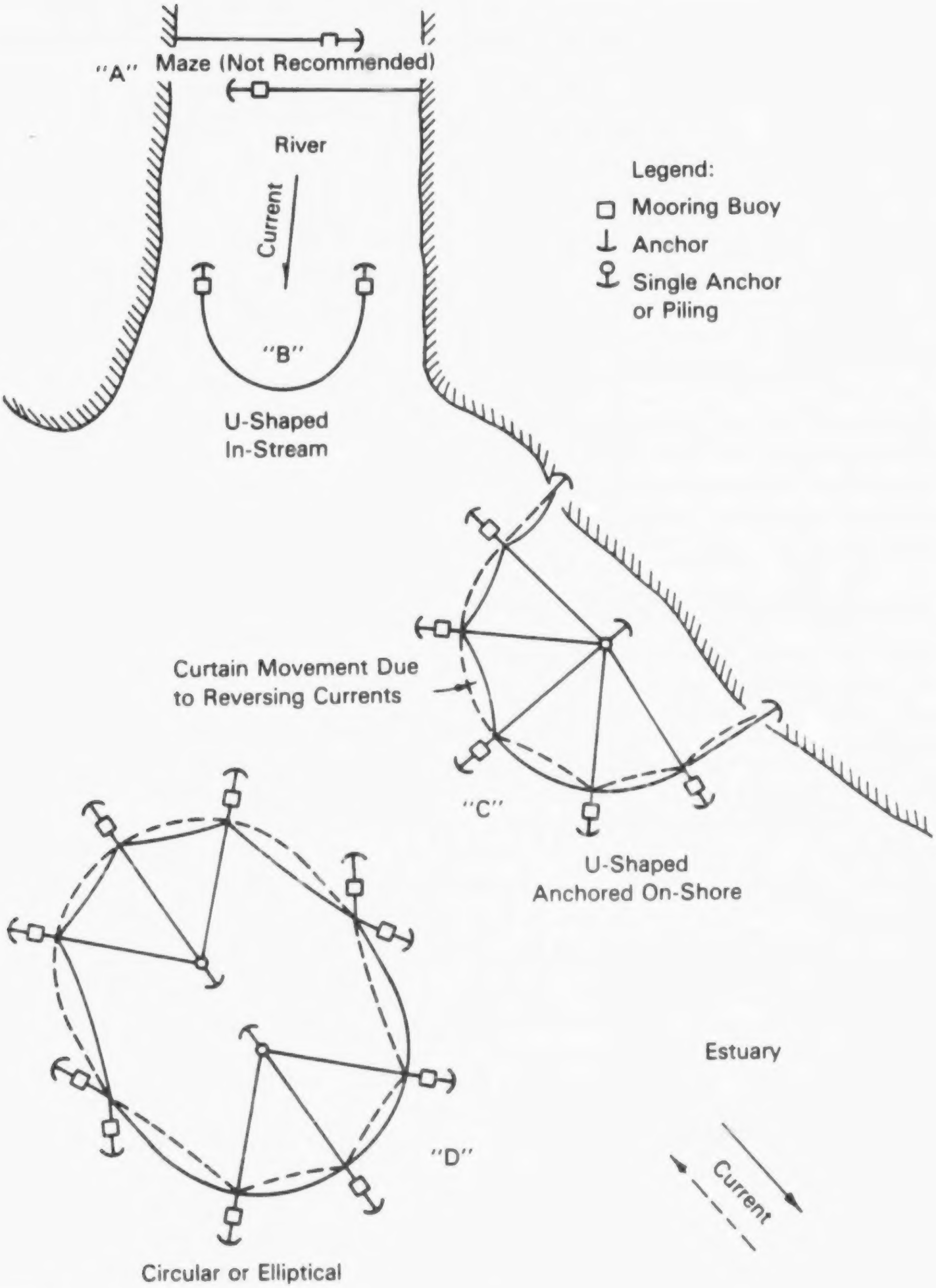
There are four basic configurations used in deploying silt curtains. These are illustrated in Figure 8-15. The maze configuration is generally not recommended because of the potential for direct flow between the separate curtain sections. The in-stream, U-shaped configuration is generally acceptable on a river where the current does not reverse. The U-shaped configuration is also suitable for operations along shores and river banks. This configuration was effectively used to contain resuspended sediments during the dredging of lead-contaminated sediments from the Mill River in Connecticut (York Wastewater Consultants, 1983). The circular or elliptical configuration is used in open waters and in tidal situations with reversing tides (JBF, 1978).

FIGURE 8-14.
CONSTRUCTION OF A TYPICAL SILT CURTAIN SECTION



Source: JBF, 1978

FIGURE 8-15.
TYPICAL SILT CURTAIN DEVELOPMENT CONFIGURATIONS



Source: Barnard, 1978

JBF Scientific Corporation (1978) performed a detailed evaluation of silt curtain effectiveness and concluded that effectiveness is related to four factors:

- Hydrodynamic conditions at the site
- Quantity and type of material in suspension
- Characteristics, construction, and conditions of the curtain
- Methods of mooring.

This JBF study should be consulted for a detailed discussion of the factors affecting performance and deployment of silt curtains.

8.1.5 Technology Selection/Evaluation

Although dredging and subsequent management of contaminated sediments is costly, in some cases it is the only viable alternative for handling contaminated sediments with the exception of the "no action" alternative. In some situations (calm, open waters or streams with low flow), in-situ methods may be used (Section 8.3). However, the long-term reliability and effectiveness of these measures is not yet known. Because of the limited number of alternatives for handling contaminated sediments, the decision is frequently not whether to dredge but how to dredge.

Factors affecting the implementability, effectiveness, and reliability of a dredging operation depend largely on the inherent capabilities and limitations of the dredging equipment. These are summarized in Tables 8-1 and 8-2.

The time required to implement dredging will depend primarily upon the availability of the dredge and the location from which it must be transported. Also important to the implementation schedule are the weather conditions (for rivers and streams, dredging should take place during periods of low flow to minimize downstream transport of contaminants) and the time it takes to obtain the necessary permits (State permits and permits required under Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbor Act).

The effectiveness of the dredging operation depends upon the precision obtainable with the dredge, types of dredge vessel controls used and the operators capabilities. Inaccuracies in dredge cut positioning and depth control, sediment sloughing, and difficulties with obstructions and debris will cause the dredging project to be less than 100 percent efficient. Recovery of contaminated sediment during normal operations using conventional (hydraulic and mechanical) dredges has been estimated to be as low as 65 percent. On the other hand implementation of double pass dredging to remove the remaining contaminated sediments generally yields a substantial amount of uncontaminated sediment which must subsequently be treated. Preplanned

TABLE 8-1. COMPARISON OF DREDGE EQUIPMENT TO SELECTION CRITERIA

	Type of Dredge										
	Mechanical			Hydraulic					Pneumatic		
	Clamshell	Dragline	Backhoe	Plain Suction	Cutterhead	Dustpan	Hopper	Portable	Air Life	Pneuma	Oozer
Typical Application											
- Volume	A,B	A,B		B,C	B,C	B,C	B,C	A,B	A,B,E	B,C,E	B,C,E
- Setting	D,E,F,G,H	D,E,F	D,E	G.H	G,H	G,H	H	E,F,G	F,G,H	F,G,H	F,G,H
Auxiliary Facilities	I,J,M	I,J.M	I	J,K,L	J,K,L	J,K,L	None	K,L,M	J,K,L,M	J,K,L,M	J,K,L,M
Depth Limitations											
- Minimum (ft)	None(4)	None(3)	None	56(3)	35(2)	5(2)	12-30(1)	11/26(5)	None	(4)	(4)
- Maximum (ft)	None(6)	50(3)	60	60	12-50(2)	60(2)	36-65(1)	15-58(5)	None	None(6)	None(6)
Precision Obtainable											
- Horizontal (ft)	1(3)	1/2	1	2-3(3)	2-3(3)	2-3(3)	10(3)	1(3)	1	2-3(3)	2-3
- Vertical (ft)	1	1	1	1	1	1	1	1	1	1	1
Rate of Production	30-600(3)	30-60(1)		25-10,000(3)	25-2,500(2)	3,500(3)	500-2,000(3)	50-1,850(5)	(9)	60-390(2)	(9)
Turbidity/ Resuspension	High(3)	High(3)	High	Low(3)	Avg.(3)	Avg.(3)	Avg.(3)	Low(3)	Low	Low(3)	Low
Availability	T	T	T	X(8)	R(8)	Q(8) X(8)	Q(8) S(x)	U	Y	V	W
Transportability	4	4	1	5	5	5	6	13(8)	4	4	4
Slurry Solids Content	in-situ	in-situ	in-situ	10-15%	10-20%	10-20%	10-20%	10-40%	10-20%	up to 80% of in-situ	30-70%

(continued)

TABLE 8-1. (continued)

Footnotes

(1) See Table 4-3, p. 54, Hand et al., 1978

(2) See Table 4-2, p. 51, Hand et al., 1978

(3) See Table 4-4, p. 59, Hand et al., 1978

(4) Determined by draft of vessel; if not vessel-mounted, there is no limiting minimum depth.

(5) See Table 1, p. 12 (Clark, 1983)

(6) Limited only by availability of support equipment (e.g., cables, winches, etc.)

(7) See Tables 4-8 through 4-11, pp. 76-79, Hand et al., 1978

(8) See Table 4-6, p. 69, Hand et al., 1978

Volume

A - Small scale, less than 1,000 cubic yards. (See Footnote 7.)

B - Medium scale, 1,000 to 200,000 cubic yards. (See Footnote 7.)

C - Large scale, greater than 200,000 cubic yards. (See Footnote 7.)

Setting

D - Narrow and/or very shallow (less than 5 feet) streams.

E - Shallow (less than 20 feet) streams and rivers, navigable by small vessels.

F - Inland lakes and ponds.

G - Inland navigable channels and lake and coastal harbors.

H - Great lakes and coastal harbors.

Auxiliary Facilities

I - Dump trucks

(Continued)

TABLE 8.1. (continued)

Auxiliary Facilities (continued)

J - Barges

K - Transport Piping

L - Settling Impoundments

M - Crane

Availability

Q - All or most owned by U.S. Army Corps of Engineers.

R - Bases in most major harbors and commercial waterways.

S - Based in some coastal and great lakes harbors.

T - Widely available in general earthwork applications.

U - Widely available from contractors and vendors.

V - Limited availability through United States distributors.

W - Not generally available in United States.

X - Generally available on inland commercial waterways.

Y - Can be fabricated.

Z - Very few operating in United States.

Transportability

1 - Dredge can be moved over existing roads "as-is" or with slight modification. (See Footnote 5.)

2 - Dredge can be moved over existing roads after disassembling to 3 or fewer pieces. (See Footnote 5.)

3 - Dredge can be moved over existing roads after disassembling to more than 3 pieces. (See Footnote 5.)

4 - Dredge head can be removed over existing roads "as-is" or with slight modification and mounted on conventional vessel or crane.

5 - Transport restricted to navigation channels (greater than 5 foot depth) due to draft.

6 - Transport restricted to deep (greater than 12 feet) navigation channels due to draft.

TABLE 8-2. COMPARISON OF DREDGE EQUIPMENT TO APPLICABILITY TO WASTE SITE CONDITIONS

	Type of Dredge										
	Mechanical			Hydraulic						Pneumatic	
Factor	Dragline	Backhoe	Clamshell	Plain Suction	Cutterhead	Dustpan	Hopper	Portable	Air Lift	Pneuma	Oozer
Vessel Draft (ft)	a	a	a	5-6	3-14	5-14	12-31	11/2	3-6	N/A	7
Hindrance to Traffic	small	small	small	significant	some	significant	small	some	some	some	significant
Maximum Wave Height	<3	<3	<3	<3	<3	<3	<7	<1	<3	<3	<3
Operation Near Structures	Yes	Yes	Yes	No	No	No	No	No	No	No	No
Operation in Open Water	limited	limited	limited	limited	limited	no	yes	no	no	limited	yes
Capable of Consolidating Sediments	yes	yes	yes	no	yes	yes	no	no	no	no	yes
Susceptable to Debris Damage	no	no	no	yes	yes	yes	yes	yes	yes	yes	yes
Suitable for Liquid or Solid Removal	solid	solid	solid	both	both	both	both	both	both	both	both

[a] Depends on draft of supporting structure. Most barges have a draft of 5 to 6 ft.

overlaps and dredgecuts controlled with modern electronic locating equipment are recommended to maximize efficiencies.

Short-term impacts from dredging are a primary concern. Resuspension and downstream transport of sediment can be minimized using low turbidity dredges, by dredging during low flow, and through the use of turbidity controls. In addition, a monitoring plan should be implemented which requires immediate cessation of dredging when resuspension or downstream transport exceeds a certain level.

Acceptance of a proposed dredging operation cannot be evaluated separately from the dredged materials handling (see Sections 10.2 and 8.2). Dredges which result in high slurry densities will facilitate good dredge materials management.

8.1.6 Cost

Unit cost associated with representative hydraulic and mechanical dredging techniques are described in Tables 8-3 and 8-4. Capital purchase cost and operating cost are given for some dredge vessels and accessories, although it is recognized that dredging will most likely be performed by specialty contractors whose rates maybe highly variable from site to site.

Cost considerations that are not included in Tables 8-3 and 8-4 are the following:

- Crane rental to launch and retrieve portable dredge vessels
- Freight and handling costs for shipping dredge equipment
- Transportation of equipment from site to site
- Insurance (hull coverage and reliability) for purchased vessels
- Storage and/or warehouse costs
- Sales tax.

Unit costs for conventional excavation equipment are included in Section 7.1.

It should be noted that the costs associated with a dredging operation may be a small part of the total costs of a contaminated sediments cleanup. Dewatering, treatment and disposal costs can be a large percentage of total capital costs. Sections 10.2 and 8.2 should be consulted for technical and cost information associated with dredged materials handling.

TABLE 8-3. UNIT COSTS FOR MECHANICAL DREDGING/EXCAVATION

Description	Unit Cost	Source
Dredge mobilization and demobilization (Add to total)	\$6,175 - 32,800 (add to total unit costs)	1
Barge-mounted dragline or clamshell, hopper dumped, pumped 1,000 feet to shore dump	\$6.00 - 8.40/yd^3	1
Bulk excavation, medium earth-piled or truck-loaded (no trucks or haul included):		
Hydraulic backhoe (crawler-mounted)		
- capacity - 1 cubic yard (45 CY/hr)	\$2.44/yd^3	1
- capacity - 3 1/2 cubic yard (150 CY/hr)	\$1.64/yd^3	1
Hydraulic backhoe (wheel-mounted)		
- capacity - 1/2 cubic yard (20 CY/hr)	\$4.49/yd^3	1
- capacity - 3/4 cubic yard (30 CY/hr)	\$3.30/yd^3	1
Clamshell		
- capacity - 1/2 cubic yard (20 CY/hr)	\$4.83/yd^3	1
- capacity - 1 cubic yard (35 CY/hr)	\$3.23/yd^3	1
Dragline		
- capacity - 1/2 cubic yard (30 CY/hr)	\$3.50 yd^3	1
- capacity - 3 cubic yard (112 CY/hr)	\$1.46/yd^3	1
For Wet Excavation:		
Clamshell or dragline	increase cost above by 100%	
Backhoe or other equipment	increase cost above by 50%	
Clamshell excavation in sheeting or cofferdam - for soft soil or sand, deduct 15% - for heavy soil or stiff clay, add 60%	\$4.08-15.35	1

(continued)

CY/hr = cubic yards per hour

TABLE 8-3. (continued)

Description	Unit Cost	Source
Mobilization and demobilization of backhoe or dragline		
- capacity 3/4 cubic yard	\$140 each	1
- capacity 1-1/2 cubic yard	\$210 each	1
Earth hauling, 12 cubic yard dump truck, 4-mile round trip, 1.6 loads/hr.	\$ 3.28/yd^3	1
Sheet piling, steel (not including wales)[1]		
15-foot deep water-depth (22 psf)		
- left in place	\$ 10.65/ft^2	1
- pulled and salvaged	\$ 7.15/ft^2	1
20-foot deep excavation (27 psf)		
- left in place	\$ 12.35/ft^2	1
- pulled and salvaged	\$ 7.70/ft^2	1
25-foot deep excavation (38 psf)		
- left in place	\$ 15.70/ft^2	1
- pulled and salvaged	\$ 8.30/ft^2	1
Pile driver, steam or air, 8750 ft-lb at 145 BPM	\$595/week (rental)	1

(continued)

psf = pounds per square foot

BPM = blows per minute

[1] In general, the ratio of exposed length of sheeting to driven length (unexposed, anchored into the soil) should be about 1:1, with 1 to 3 feet of freeboard above the water surface (Staples, G., JRB Associates, personal communication, 1980).

TABLE 8-3. (continued)

Description	Unit Cost	Source
Corrugated metal pipe (not including excavation and backfill):		
24-inch diameter, 14 gauge	$ 19.15 linear foot	1
30-inch diameter, 14 gauge	$ 24/linear foot	1
36-inch diameter, 12 gauge	$ 40/linear foot	1
48-inch diameter, 12 gauge	$ 52/linear foot	1
60-inch diameter, 10 gauge	$ 81/linear foot	1
Stone rip-rap (filter stone dumped from trucks for outlet protection)	$ 21/yd^3	1
Pump, centrifugal gas:		
1-1/2-inch, 4 MGPH	$ 48/week (rental)	1
2-inch, 8 MGPH	$ 64/week (rental)	1
3-inch, 15 MGPH	$ 80/week (rental)	1
6-inch, 90 MGPH	$240/week (rental)	1
Pump, submersible, electric:		
1-1/4-inch, 55 GPM	$ 50/week (rental)	1
1-1/2-inch, 83 GPM	$ 67/week (rental)	1
2-inch, 120 GPM	$ 85/week (rental)	1
3-inch, 300 GPM	$115/week (rental)	1
4-inch, 560 GPM	$150/week (rental)	1
6-inch, 1590 GPM	$400/week (rental)	1

(continued)

MGPH = thousand gallons per hour
GPM = gallons per minute

TABLE 8-3. (continued)

Description	Unit Cost	Source
Pump hose with coupling		
2-inch, 20-foot section	\$ 17/week (rental)	1
3-inch, 20-foot section	\$ 23/week (rental)	1
4-inch, 20-foot section	\$ 33/week (rental)	1
6-inch, 20-foot section	\$ 65/week (rental)	1
8-inch, 20-foot section	\$ 83/week (rental)	1
Silt curtain		
8-inch diameter float, 10-foot skirt	\$ 3/ft^2	2
vinyl-coated	\$ 25/ft^2	2
urethane-coated	\$ 35/ft^2	2
Hauling of contaminated sediments	\$ 3-4/loaded mile	3
Disposal of contaminated sediments	\$200/ton	3

Sources: 1. Godfrey, 1984

2. Kepner Plastic Fabricators, Torros, CA, personal communication, March 1985.

3. West, R. Cecos International, Niagara Falls, NY, personal communication, March 1985.

TABLE 8-4.
1985 HYDRAULIC DREDGE UNIT COSTS

Item	Unit Cost	Source
Vaughn Lagoon Pumper with 75 hp motor, complete platform assembly including winch	$20,000 - 50,000	1
Mud Cat Dredge Models SP-810, with 10-foot boom and submerged pump; less accessories	$84,500	2
Mud Cat Dredge Model SP-815, with 15-foot boom and submerged pump; less accessories	$94,900	2
OPTIONAL EQUIPMENT (FACTORY-INSTALLED)		
Air conditioned cab	$ 1,900	2
Auger wheels (for use in lined ponds)	$ 950	2
Auger cage assembly (extra protection for lined ponds)	$ 950	2
Anodes (for salt water application)	$ 550	2
ACCESSORY EQUIPMENT PACKAGES		
Discharge pipe package, 1,500 feet of 6-inch polyethylene pipe	$21,486	2
Discharge pipe package, 1,500 feet of 8-inch polyethylene pipe	$25,909	2
Harnessing hardware	$ 4,466	2
Service boat and motor	$ 3,301	2

(continued)

TABLE 8-4. (continued)

Item	Unit Cost	Source
EXTRA DISCHARGE PIPE AND CONNECTORS		
Carrier pipe, 6-inch x 19 feet, male and female couplings, including gasket, float bands and links not included	$149	2
6-inch ring band lock	$ 8.35	2
Carrier pipe, 8-inch x 19 feet, male and female couplings, including gasket, float bands and links not included	$189	2
8-inch ring band lock	$ 8.61	2
BOOSTER PUMP AND EQUIPMENT		
Booster pump, skid-mounted, including connector fitting kit	$30,100	2
Booster pump, skid-mounted, including connector fitting kit	$30,100	2
OPERATING COSTS		
Fuel $1.20/gal x 5 gal/hr	$ 6/hr	2
Lubricants: Engine 0.10/gal hr Hydraulics 0.07 gal/hr Grease 0.06 lb/hr	$ 1.20/hr	2
Repairs/parts	$ 1.50/hr	2
Two operators and fringe benefits	$20/hr	2

Other cost considerations not included: crane rental to launch and retrieve Mud Cat Dredge, transportation of equipment from site to site, administrative and sale costs, storage and/or warehouse costs, service vehicle cost, per diem, sales, tax, interest on capital.

Source: 1. Vaughn Co., Inc., Montesano, WA, personal communication, 1985
2. Sherman, L., Mud Cat Division of National Car Rental System, Inc., Fort Lee, NJ, personal communication, 1985

8.2 Dredge Material Management

Contaminated dredge spoil management includes methods for dewatering, transporting, treating, and disposing of contaminated sediments after dredging. The most technically and economically effective strategy to handle contaminated sediments removed from a given dredge site will depend on many site-specific variables, which include the following:

- Method of dredging used
- Method of spoil transport - pipeline vs. truck or hopper barge
- Physical nature of removed spoil - consistency (solids/water content) and grain size distribution
- Volume of dredge material
- Nature and quantity of contamination - physical and chemical characteristics of contaminants and hazard/toxicity level of contamination
- Proximity of acceptable treatment, storage, or disposal facilities
- Available land area for construction of dewatering and treatment facilities.

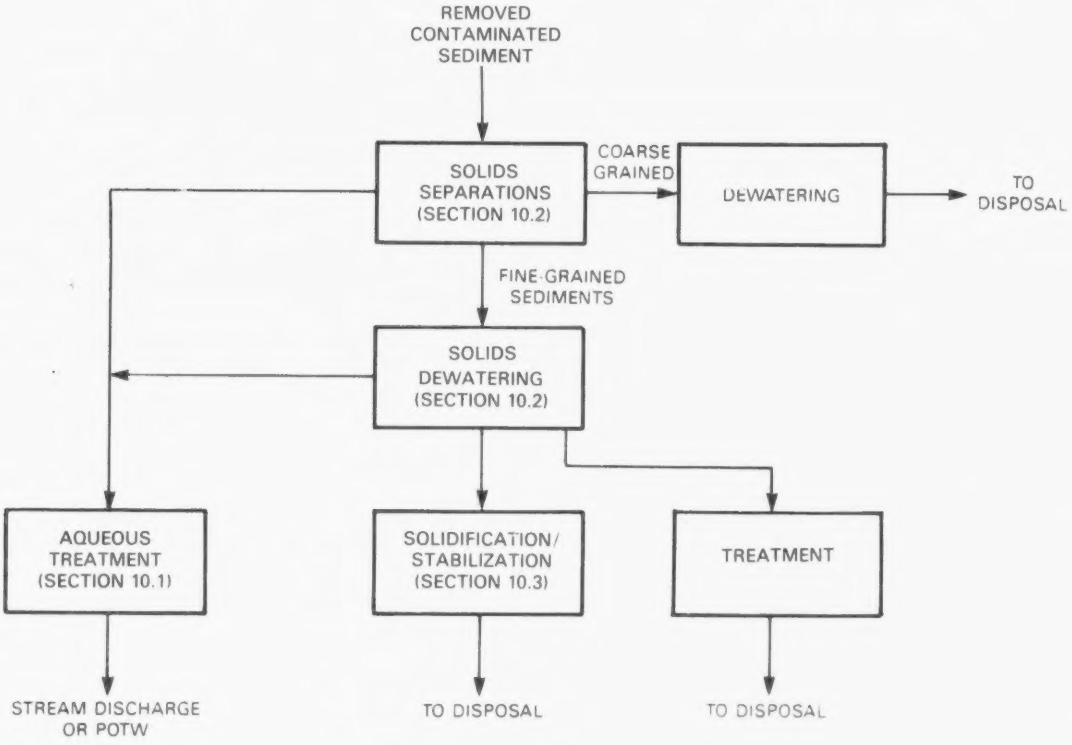
There are several well-established techniques for the management of uncontaminated dredge spoil. Techniques for managing contaminated dredge spoil, however, are influenced by the hazardous nature of the spoil material. Special consideration must be given to handling these sediments in a safe, efficient manner.

Figure 8-16 summarizes the major activities that are undertaken in order to manage dredged materials. Many of the activities apply to hazardous waste contaminated soils and sludges as well as sediments, and they are discussed in detail in other sections of this Handbook; Figure 8-16 also refers the reader to the appropriate sections.

Costs presented in Tables 8-3 and 8-4 do not include the cost of dredge material management (see Sections 10.2 and 8.2). These costs, which may include dewatering, solids and aqueous waste treatment and transport to a disposal site may far exceed the costs associated with the actual dredging operation. This point is illustrated by cost estimates which have been developed for dredging and sequent handling of contaminated sediments from Waukegan Harbor, Illinois.

The Feasibility Study for Waukegan Harbor examined a broad range of alternatives for handling PCB contaminated sediments (Ch_2M-Hill and Ecology and Environment, 1983). The costs estimates (order of magnitude) for one such alternative (not a recommended alternative) involving dredging 10,900 cubic

FIGURE 8-16.
OVERVIEW OF DREDGED MATERIAL MANAGEMENT


REMOVED CONTAMINATED SEDIMENT
SOLIDS SEPARATIONS (SECTION 10.2)
COARSE GRAINED
DEWATERING
TO DISPOSAL
FINE-GRAINED SEDIMENTS
SOLIDS DEWATERING (SECTION 10.2)
AQUEOUS TREATMENT (SECTION 10.1)
SOLIDIFICATION/ STABILIZATION (SECTION 10.3)
TREATMENT
STREAM DISCHARGE OR POTW
TO DISPOSAL
TO DISPOSAL

yards of sediments from a harbor slip and dewatering and treating the contaminated sediment are summarized in Table 8-5. Total present worth cost (expressed in 1985 dollars) for the alternative is $11,137,380 or $1024/cubic yards. The elements of the remedial action are briefly described below.

A sediment dispersal control device, consisting of a double silt curtain or sheet piling, would be installed across the mouth of the harbor slip. 10,900,000 cubic yards of sediments with greater than 50 ppm PCBs would be removed with a hydraulic dredge (cutterhead pipeline suction depth dredge) and the sediment slurry pumped through a pipeline to the initial solids dewatering lagoon (described below). Because the hydraulic dredge cannot penetrate the area of deep contaminated sand and silt near the OMC outfall, a clamshell dredge would be used to remove this material.

This deep dredging would be performed inside a single sheet pile cofferdam. The solids would be loaded onto trucks and transported to an initial solids dewatering lagoon. The water level inside the cofferdam would be kept lower than outside to cause water flow toward the contained area. The removed water would be routed to a water treatment plant for suspended solids and PCB removal (to 1 ppb PCBs), then discharged to the harbor or to a sanitary sewer.

Solids would be dewatered in a clay-lined dewatering lagoon constructed on OMC property. Volatilization would be controlled by covering the filled lagoon surface with organic sludge. The supernatant would be continuously decanted and routed to a 1,500-gpm water treatment plant to remove suspended solids and dissolved PCBs down to 1 ppb before being discharged. After dredging activities are completed, rainwater and leachate water would be treated by the 1,500-gpm water treatment plant for the duration of the dewatering process.

Solids would be removed from the lagoon by dragline about 2 months after dredging activities are completed, loaded into trucks, and transported to a batch treatment plant.

The solids would be fixed at the batch plant by adding portland cement, Locksorb, or another fixing agent to hydrate the excess water. The mix would then be transported to curing cells (described below). The fixed solids would cure until they were non-flowable. This is expected to take about 1 day. The fixed solids would be removed from the curing cells by front end loaders for transportation by truck to an approved disposal site.

Initial Solid Dewatering--A 24,000 cubic yard lagoon would be required to dewater sediments from the slip. The capacity of the lagoon is based on 2 feet of freeboard and 8 feet of storage. The lagoon would have a clay liner system consisting of the following:

- A 6-inch-thick, compacted soil-cement layer
- A 1-ft-thick, compacted clay liner with a permeability less than 10^{-8} centimeters per second

TABLE 8-5. COST ESTIMATE WAUKEGAN HARBOR SLIP DREDGE-DEWATER IN LAGOON-FIX-DISPOSE

Description	1985[1] Capital Costs	1985[1] Present Worth of O&M Costs	1985[1] Present Worth
Mobilization	$ 622,200	$ 0	$ 622,200
Health and Safety Requirements	342,210	0	342,210
General Site Preparation	300,730	0	300,730
Sediment Dispersal Control	62,220	0	62,220
Dredging	62,220	0	62,220
Localized Cofferdam and Dewatering	409,430	62,220	466,650
Dredging of Deep Sand and Silt	20,740	0	20,740
Initial Solids Dewatering--Lagoon	1,628,090	10,370	1,638,460
Water Treatment Plant and Water Disposal	1,306,620	62,220	1,368,840
Solids Removal	62,220	0	62,220
Fixation	1,638,460	0	1,638,460
Transportation and Disposal	734,270	0	734,270
Engineering, Legal, and Administration	1,223,660	20,740	1,244,400
Subtotal	2,519,910	51,850	2,571,760
Total	$10,929,980	$207,400	$11,137,380

(1)(Costs were updated to $1985 using ENR construction cost indices for 1983 and 1985).

Source: CH_2M-Hill and Ecology and Environment, 1983.

- A 1-ft-thick sand or gravel layer with pipe underdrains to collect any PCB-contaminated water that may penetrate the clay liner

- A 1-ft-thick compacted clay liner with a permeability less than 10^{-8} centimeters per second for additional protection against percolation of PCB-contaminated water.

Curing Cells--Three 1,400-cubic yard curing cells would be required to cure the fixed solids from the batch plant. The capacity of the curing cells is based on 1 foot of freeboard and 4 feet of storage. The earth-lined cells would have the same clay liner system as described for the lagoons. In addition, the curing cells would have 2-ft-thick, 5-ft-high concrete walls to divide the earth-lined area into three compartments.

Temporary Storage Requirements--Dredged solids would require temporary storage in a lagoon for dewatering. Dewatering is expected to take 2 months after dredging activities are completed. When the solids are removed from temporary storage, they would be fixed and would require an additional day of temporary storage for curing. After curing, they would be disposed of in a licensed chemical waste landfill.

Water Treatment--Slurry water from dredging activities would be treated before being discharged to remove PCB down to 1 ppb or less.

Water treatment would consist of:

- Coagulation/sedimentation to coagulate and settle fine sediments
- Sand filtration to remove suspended solids
- Carbon filtration to remove soluble PCBs
- A clearwell to monitor PCB levels before the water is discharged

The water treatment system would be a "package plant," of factory-constructed modules, with a capacity of 1,500 gpm.

8.3 In-Situ Control and Containment Measures

In responding to a situation where bottom sediments are contaminated with hazardous substances, it is sometimes technically infeasible or economically unreasonable to remove all of the contaminated material from its location in the watercourse. If removal is determined to be an unacceptable singular remedial response, in-situ control and containment measures are often considered. These measures are intended to reduce dispersion and leaching of a hazardous substance to other areas in the water body. They may be temporary or permanent response measures.

The use of in-situ methods for permanent containment of hazardous waste contaminated sediments is neither widely practiced nor well-demonstrated and consequently these methods will not be discussed in detail. Laboratory and pilot scale testing is likely to be required before these methods can be implemented at a particular site. Permanent containment methods may include use of caps, dikes or in-situ grouting. Temporary containment usually involves the use of dikes or berms, although capping materials can be used on a temporary basis as well.

8.3.1 Retaining Dikes and Berms

8.3.1.1 General Description

Retaining dikes and berms include earthen embankments, earth-filled cellular and double sheet pile walls, water inflated dams and other materials which can be used to minimize transport of contaminated sediments.

8.3.1.2 Applications/Limitations

Retaining dikes or dams can be constructed perpendicular to the direction of stream flow, or downstream of a dredging operation in order to prevent suspended particulate matter from flowing downstream. This type of dike creates the effect of a holding pond or reservoir, which prevents flow downstream and also promotes the settling of fine particles. The damming creates deeper areas where water velocity is slower and allows more time for small particles to settle. Retaining dikes used for this application are limited to streams with low flow. A water inflated dam constructed from reinforced urethane can also be used for this purpose.

Retaining dikes can also be constructed parallel to a river or stream bank to isolate contaminated deposits from the deeper river channel. When stabilized with vegetation, capping or some other method, these dikes can provide permanent containment.

8.3.1.3 Implementation Considerations

Earthen dikes can be constructed quickly and easily using readily available earth moving equipment such as bulldozers, front-end loaders or mechanical dredging equipment.

Water-inflated dams are constructed by securing the ends to steel piles or deadmen on shore and weighting the bottom of the sediments under the dam to provide a tight seal.

Construction of dams using sheet piling is described in Section 8.1.4.1.

8.3.2 Cover Methods

8.3.2.1 General Description

A wide variety of materials can be used to cover contaminated sediments in order to minimize leaching of contaminants and prevent erosive transport of contaminated sediments. Cover materials include inert materials such as silt, clay or sand and active materials or additives which react with contaminants to neutralize or otherwise decrease inherent toxicity. Potentially applicable active cover materials include: limestone and greensand for neutralization; oyster shells or gypsum for metals precipitation; ferric sulfate for both precipitation and base neutralization; and alum for base neutralization (Hand, 1978).

8.3.2.2 Applications/Limitations

Cover materials have application for temporary or permanent containment for hazardous waste constituents. Their use is generally limited to protected open waters where bottom currents and flow velocity are generally not sufficient to erode the cap. Some of the active materials can be applied together with inert cover material to treat and contain the sediments.

The active covering strategy differs from the inactive covering strategy because each waste constituent must be evaluated on a case-by-case basis, whereas the performance of the inert materials is not as strongly affected by the waste constituents.

A major limitation with the use of these methods is that their feasibility and effectiveness has not been demonstrated for treatment/containment of hazardous waste contaminated sediments. Covering methods have been applied to contaminated sediments in Stamford-New Haven Harbor, Connecticut, (Morton, 1980) and the Port of Rotterdam in the Netherlands (Van Leeuwan, Kleinbloesem and H.J. Groenewegan, 1983), but long-term reliability of these actions is not yet known.

8.2.2.3 Design Considerations

The behavior of inert cover materials depends largely on the particle size and cohesiveness of the cover materials, and the dynamics of bottom flow. In determining the most appropriate material (sand, clay or silt) to use for capping, three factors need to be considered:

- Susceptibility to scour and resuspension - The susceptibility of cover material to scouring depends on: particle size, shape, and size distribution; dynamics of flow; shape of the bottom surface; angle of repone of the particles; and degree of cohesiveness of the cover material. Generally, the most important of these parameters are the cohesiveness of the cover material and the roughness factor associated with clumps of cohesive sediments. Water currents sufficient to erode or scour cohesive sediments are generally greater than those for noncohesive sediments of approximately the same grain size.

The rate and extent of scour depends upon cohesive strength rather than on the properties of individual particles. Thus sand will be more subject to erosion than silt or clay. However, if there are clumps of cohesive material in the cover shear stress generated by bottom currents acting on these rough surfaces will tend to cause erosion (Hand, 1978; Morton, 1980).

- Ability to withstand leaching - The ability of cover material to withstand leaching is believed to be directly related to permeability. The coarser and more permeabie the material the more prone it is to leaching. Therefore clays will exhibit much slower rates of leaching than sand.
- Effects on benthic organisms - The ability of benthic organisms to recolonize a capped area without significant bioaccumulation depends on: the type of cover material; the similarity to natural surrounding sediments; thickness of cover; and the potential for leaching. Also, the cap must be sufficiently thick to prevent burrowing.

With active cover materials, the criteria for selection are different. If active cover materials are to be used successfully, they must remain in place long enough to react with and treat the contaminants. Limestone and gypsum are pozzolanic in nature and tend to form a thick, cement-like cover that is resistant to erosion. Ferric sulfate, alum, and alumina are very fine-grained and can be expected to behave like clays. Greensand and oyster shells will probably scour most easily, and it may be necessary to mix these materials with a more stable inert cover material (Hand et al., 1978).

8.3.2.4 Implementation

The method of emplacing active or inert cover materials is important to the overall effectiveness of the cover system. For this reason, the method should be selected carefully. There are basically three methods for emplacing cover materials, as follows:

- Point dumping
- Pumpdown methods
- Submerged difuser systems.

a. Point Dumping

Point dumping of cover material from barges, scows, or hopper dredges is a relatively straightforward and rapid approach to conducting a capping operation. However, use of barges, scows, and hopper dredges for point dumping results in a high degree of turbidity and dispersion relative to other capping methods, particularly where low moisture, fine-grained silts and clays are used for capping. Also, a deep draft precludes the use of these methods in shallow waters.

Most modern barges are the "clamshell" type, in which dumping is accomplished by hydraulically opening the entire hull in a clamshell-like motion. Most scows are divided into several compartments (usually six) with each compartment having one pair of bottom opening doors. The scowman can thereby empty the compartments selectively. Dump scows and barges have little or no navigational equipment aboard; the basic power and navigational capabilities reside with the tugboat (Johanson et al., 1976).

Barges and scows have capacities ranging from 2000 to 4000 cubic yards and a loaded draft ranging from 7 to 20 feet. Draft limitations preclude the use of barges and scows in shallow water. Seven to 8 feet of water beyond the loaded draft is required for point dumping (Johanson, 1976).

The hopper dredge is similar to a scow insofar as it is divided into separate hoppers and point dumping is accomplished by gravity-dumping through hydraulically operated gates in the bottom of the hoppers. Unlike barges and scows, however, the hopper dredge is self-propelled and the navigational controls are located on the vessel. No anchors or mooring system is required, and the operation is very efficient. Hopper dredges can operate in rough open waters, with relatively strong currents; a sea-going hopper dredge can be expected to hold a clearly defined position for 1 to 2 hours in currents of up to 4 knots (Johanson, 1976).

b. Pumpdown Methods

Pumpdown of the cover material from barges, scows, or hopper dredges would create substantially less turbidity and/or resuspension of contaminated sediments than point dump methods, provided that the discharge pipe is situated close to the bottom.

Conventional pumpdown barges pump the capping material from a scow, barge, or land-based storage area down a discharge pipe whose termination is set close to the bottom. For use in a capping operation, the pumpdown barge would be moored or spudded into place and scows or barges containing the cover material would be tied along side. Equipment on the unloading barge would then be swung over the hopper of the scow and the load hydraulically pumped out.

As with point-dumping methods using barges and scows, the pumpdown method is limited to use in relatively calm waters and is not applicable in shallow waters due to the depth of water that is drawn by the vessel. The loaded draft of barges typically ranges from 7 to 20 feet (Johanson, 1976).

A hopper dredge pumpdown system would be more applicable than a pumpdown barge in unprotected waters where conditions might restrict the use of barge-mounted systems. The use of the hopper dredge allows for increased efficiency and more control over the capping operations. However, availability and draft restrict the use of the hopper dredge pumpdown system (Johanson, 1976).

Use of a hopper dredge in a pumpdown capacity would require modification of the existing equipment to provide a pumpdown (rather than pumpup) capability through the dragarm (See description of hopper dredge in Section 8.1). The maximum effective depth for the pumpdown option would be 36 to 60 feet, depending on the length of the dragarm. This method could be used in deeper waters, although the effectiveness would greatly decrease. Since the draghead will nominally be about 10 feet wide, precise navigation would be difficult, especially if full cover is required (Johanson, 1976). Nevertheless, sophisticated navigation control systems are available that permit precision navigation. A hopper dredge modified for pumpdown was used together with a diffuser head for a capping operation in the Port of Rotterdam (Van Leeuwan, 1983).

The two pumpdown methods described above present a clear advantage over point dumping, insofar as turbidity and resuspension of contaminated bottom sediment is largely reduced with these methods. The methods are, however, considerably slower, and in the case of barge mounted and hopper dredge pumpdown systems, require very exacting navigational controls and monitoring to ensure complete coverage (Johanson, 1976).

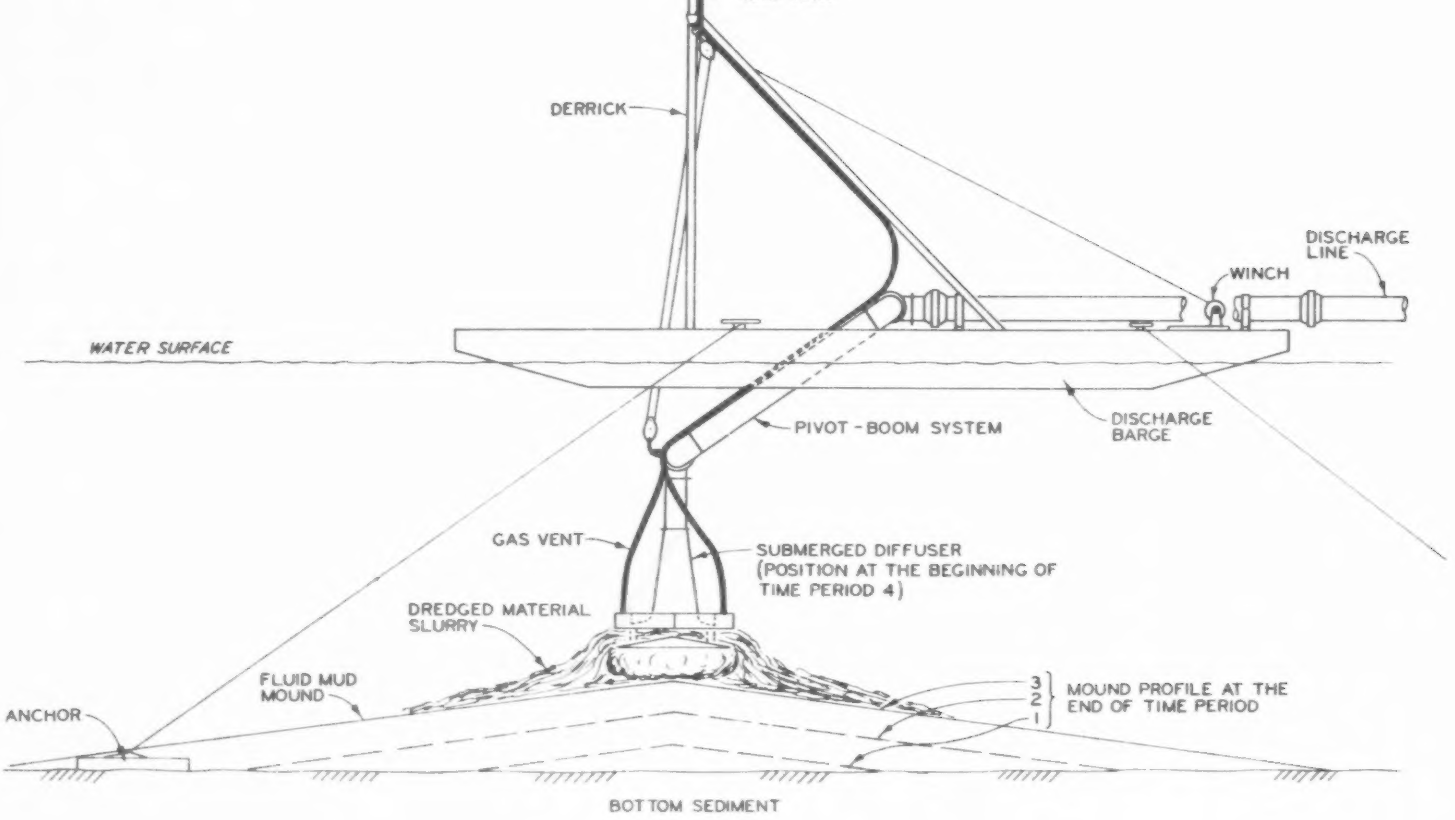
c. Submerged Diffuser System

The submerged diffuser system is one of the most effective methods for controlling the placement of cover material. The advantages of this system include increased control over the location of cover, decreased scouring of the bottom area, and less turbidity in the area.

The primary purpose of the diffuser head is to reduce the velocity and turbulence associated with the discharged cover material. This is accomplished by routing the flow through a vertically oriented axial diffuser. The diffuser head, illustrated in Figure 8-17, operates on the principle that radial divergence of the flow will slow the discharge velocity to acceptable levels. By varying the height of the discharge above the bottom as well as the discharge velocity, impact velocity and thickness of cover can be controlled (Barnard, 1978 and Hand et al., 1978).

A hydraulic barge/pipeline system would be used with the diffuser to provide both the support and the capability for lowering the diffuser to within 3 feet of the bottom at the beginning of the disposal operation. The diffuser discharge would be raised or lowered to the desired level by a

FIGURE 8-17. SUBMERGED DIFFUSER SYSTEM, INCLUDING THE DIFFUSER AND DISCHARGE BARGE



Source: Barnard, 1978

derrick on the barge, as shown in Figure 8-17. The submerged diffuser can be used to depths of about 100 feet under favorable conditions. Flow characteristics after leaving the diffuser will depend on the type of cover material, currents, pipe size, flow rate, and the height of the diffuser above the bottom (Barnard, 1978 and Hand et al., 1978).

8.3.3 Surface Sealing

8.3.3.1 General Description

Cement, quicklime, or other grouting materials can be applied to the surface of or mixed with bottom sediments to create a seal which minimizes leaching and erosive transport of contaminated sediments.

Grouts may be applied to the surface of bottom sediments using a number of approaches. These methods can generally be divided into two categories: those which involve storm diversion and those which do not.

There are essentially two approaches to sealing or stabilizing bottom sediments following stream diversion. The first is to pneumatically apply a layer of concrete (shotcrete) or grout to form a surface seal. The Olin Chemical Group pneumatically applied a 3-inch layer of concrete to the bed of the North Fork of the Holston River in Virginia after the bulk of the mercury-contaminated sediments had been removed (Brown, 1982).

The second method is to mix concrete, quicklime, or a grout with the contaminated sediments in order to stabilize the sediments. The stabilizing agent is applied to the surface and mixed with the contaminated sediments using rubber-tire or crawler-type rotor or trencher mixing equipment. The Japanese have developed a soft ground crawler vehicle (the Soil Limer) that is designed to crawl freely on soft ground and stabilize the ground by continuously and uniformly mixing the soft soil with slaked lime or cement-based solidification agents. The Soil Limer is equipped with a pair of caterpillar tracks that consist of a pair of pontoons wound with light-metal caterpillar bands by means of special rings. Contact pressure is light and the developer claims that it can float. The mixing unit is suspended between two pontoons. Both trencher and rotor types are available. The depth of mixing can be adjusted with a hydraulic cylinder; mixing to depths of 6.5 feet is possible. The tracks can then be elevated and the vehicle can be used for compaction. The machine can be disassembled into three parts for transportation (Yamanouchi, 1978; Nissan Hodo, Co. Ltd., undated).

Following completion of the sealing or stabilizing operation, the sediment bottom can be restored to its natural grade and sediment composition in an effort to restore the habitat for benthic organisms.

Concrete and quicklime have been used in a number of cases to stabilize contaminated sediment by these methods. The Japanese have used the Soil Limer in numerous cases, but no information is available on leaching of soil stabilized using this method.

Grouts and sealants can conceivably be applied to cover or cap atop contaminated sediments or spilled material without diverting stream flow.

Methods that have been used for applying concrete underwater include: concrete pumps and grouting preplaced aggregate. The diffuser head could also be used for this purpose.

Mobile concrete pumps, which may be barge mounted or used on shore, are widely used for placing concrete underwater. The mobile unit has a variable radius boom and may be used economically at either large or small sites.

Grouting of preplaced aggregate is a method which may be used in flowing streams. A coarse aggregate or combination of several types of aggregate are preplaced in forms. Grout made of cement, sand, and water can then be forced through pipes to fill the voids in the aggregate (Portland Cement Association, 1979).

The U.S. Army Engineers Waterways Experiment Station proposed the use of a modified diffuser head for use in applying cement cover on the bottom. A diffuser device which would lay the grout down in even bands would be most useful. The diffuser head could conceivably be used to apply concrete, bentonite, silicic or other grout types (Hand, 1978).

8.3.3.2 Applications/Limitations

Surface sealing methods which involve the use of stream diversion are limited to shallow waters with a low flow velocity, where diversion can be accomplished cost-effectively. The major advantage of this method is that it is unlikely to stir up the sediments and create downstream contamination. Stream diversion also simplifies the application of grouts or sealant materials.

Sealing methods which do not employ diversion are applicable to deep open water, where bottom currents are not sufficient to erode the cap. These methods will provide less resuspension of bottom sediments than in-situ injection methods (Section 8.3.4). Also sealing methods such as concrete pumps can potentially be used in confined areas not accessible to barge-mounted injection systems. However, the grout or sealant may impact the water column during application, application methods would be slow, and it may be difficult to obtain complete coverage.

Use of the diffuser head for surface sealing requires further investigation. Placement of the diffuser would be difficult, though it could be checked by remote television or perhaps by divers, depending on water clarity and on the material being covered. A hydraulic crane or other rigid positioning system might be used to provide accurate placement and control of the diffuser head. The diffuser head could conceivably be used to apply concrete, bentonite, silicic, or other grout types. Application rates using the diffuser head are expected to be slow.

8.3.4 In-Situ Grouting

8.3.4.1 General Description

The stabilization of contaminated sediments can be achieved through the injection of grouting materials into sediments. Chemical injection using clay-cement or quicklime has been used widely, particularly by the Japanese, for stabilizing bottom sediments prior to the construction of port and harbor structures. A commonly used Japanese method for grouting with clay-cement is the Deep Cement Mixing Method. The system shown in Figure 8-18 consists of a number of injection pipes mounted on a barge; the injection pipes are connected to mixing pipes that enter the sediments. Similar equipment is available for deep mixing with quicklime. The process is completed by lowering the operating-mixing apparatus (mixing blades are located within the individual shafts) to the required depth and injecting a cement or lime-based slurry into the sediments. The mixing blades are then reversed and the shafts are removed and relocated (Takenaka Doboku, Co. Ltd., undated).

A number of other types of grout injection and mixing apparatus are available. Continuous mixing apparatus are available and eliminate the need to continuously raise, relocate and lower the mixing apparatus.

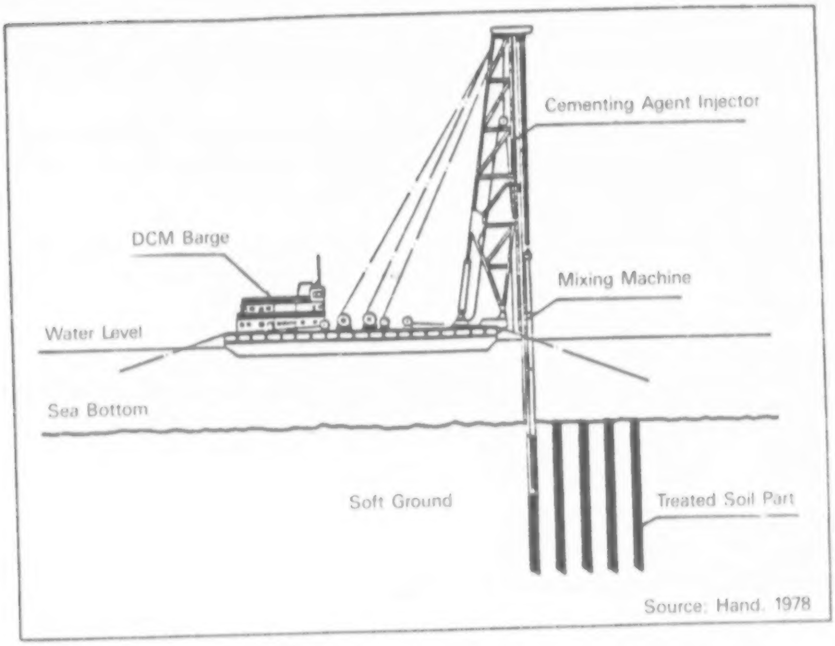
8.3.4.2 Applications/Limitations

Theoretically, in-situ grouting methods could be used to stabilize sediments to depths of about 80 to 130 feet below sea bottom (Takenaka Doboku, Co. Ltd., undated). However, the feasibility and reliability of these methods for contaminated sediments has not been demonstrated. The use of in-situ methods is restricted by barge orientation, which limits offshore activity to calm waters and periods of good weather. Injection may result in considerable resuspension of sediments.

8.3.5 Technology Selection/Evaluation

In-situ methods have potential use as an interim or emergency measure until dredging can be undertaken or as a primary remedial action where it is determined to be more cost-effective than removal. The biggest advantages to the use of in-situ methods are that they are generally much less costly than dredging, eliminate the need for dredged material management, and minimize

FIGURE 8-18. FIXATION BY DEEP CHEMICAL MIXING METHOD



Source: Hand, 1978

resuspension of contaminated sediments. Many in-situ methods also have the advantage that they can be implemented quickly and beneficial results will be almost immediate.

In-situ methods for containing hazardous wastes are a relatively new technology, and the long-term effectiveness and reliability of these methods are not well known. EPA, State Officials, and responsible parties interested in implementing in-situ methods will need to conduct a detailed site investigation and laboratory- or pilot-scale studies to determine the suitability of a particular site for in-situ containment. Given the existing state-of-the-art, this will result in considerable costs and project delays. For example, contaminated sediment boundaries and bottom currents must be carefully mapped prior to undertaking capping. Studies may be required to determine the leachability of contaminants through the capping material. Up-to-date performance data on such sites as Rotterdam Harbor, Netherlands; Stamford-New Haven Harbor, Connecticut; and Holston River, Virginia, where in-situ methods have been undertaken should be obtained. Data from these sites should be evaluated to determine the susceptibility of the caps and covers to leaching, effects on benthic organisms, and the relevance of these data to the proposed project.

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SECTION 9

IN-SITU TREATMENT

One alternative to waste excavation and removal and conventional pump and treat methods is to treat the wastes in-situ. In-situ treatment entails the use of chemical or biological agents or physical manipulations which degrade, remove, or immobilize contaminants; methods for delivering solutions to the subsurface; and methods for controlling the spread of contaminants and treatment reagents beyond the treatment zone. In-situ treatment processes are generally divided into three categories: biological, chemical, and physical. In-situ biodegradation, commonly referred to as bioreclamation, is based on the concept of stimulating microflora to decompose the contaminants of concern. In-situ chemical treatment involves the injection of a specific chemical or chemicals into the subsurface in order to degrade, immobilize, or flush of the contaminants. Physical methods involve physical manipulation of the soil using heat, freezing or other means. In many instances a combination of in-situ and above-ground treatment will achieve the most cost-effective treatment at an uncontrolled waste site.

In-situ treatment technologies are not as developed as other currently available technologies for restoring contaminated aquifers. However, there are some in-situ treatment technologies that have demonstrated success in actual site remediations. In addition, most of the methods are based on standard waste treatment technologies and are conceptually applicable as in-situ treatment methods. Applicability of in-situ methods must generally be determined on a site-specific basis using laboratory- and pilot-scale testing.

The following is a summary of promising in-situ treatment technologies. For a more thorough description of the technologies and the factors involved in the selection of an appropriate in-situ treatment approach, the reader is referred to the literature, particularly Review of In-Place Treatment for Contaminated Surface Soils (USEPA, 1984a) and Evaluation of Systems to Accelerate Stabilization of Waste Piles or Deposits (USEPA, 1985).

9.1 Bioreclamation

9.1.1 General Description

Bioreclamation is a technique for treating zones of contamination by microbial degradation. The basic concept involves altering environmental conditions to enhance microbial catabolism or cometabolism of organic contaminants, resulting in the breakdown and detoxification of those contaminants. The technology has been developed rapidly over recent years, and bioreclamation appears to be one of the most promising of the in-situ treatment techniques.

Considerable research conducted over the past several decades has confirmed that microorganisms are capable of breaking down many of those organic compounds considered to be environmental and health hazards at spill sites and uncontrolled hazardous waste sites; laboratory, pilot, and field studies have demonstrated that it is feasible to use this capability of microorganisms in-situ to reclaim contaminated soils and groundwater.

Microbial metabolic activity can be classified into three main categories: aerobic respiration, in which oxygen is required as a terminal electron acceptor; anaerobic respiration, in which sulfate or nitrate serves as a terminal electron acceptor; and fermentation, in which the microorganism rids itself of excess electrons by exuding reduced organic compounds.

The bioreclamation method that has been most developed and is most feasible for in-situ treatment is one which relies on aerobic (oxygen-requiring) microbial processes. This method involves optimizing environmental conditions by providing an oxygen source and nutrients which are delivered to the subsurface through an injection well or infiltration system to enhance microbial activity. Indigenous microorganisms can generally be relied upon to degrade a wide range of compounds given proper nutrients and sufficient oxygen. Specially adapted or genetically manipulated microorganisms are also available and may be added to the treatment zone.

Anaerobic microorganisms are also capable of degrading certain organic contaminants. Methanogenic consortiums, groups of anaerobes that function under very reducing conditions, are able to degrade halogenated aliphatics (e.g., PCE, TCE) while aerobic organisms cannot. The potential for anaerobic degradation has been demonstrated in numerous laboratory studies and in industrial waste treatment processes that use anaerobic digesters or anaerobic waste lagoons as part of the treatment process. Using anaerobic degradation as an in-situ reclamation approach is theoretically feasible.

9.1.2 Applications and Limitations

The feasibility of bioreclamation as an in-situ treatment technique is dictated by waste and site characteristics. More specifically, those factors which determine the applicability of a bioreclamation approach are:

- Biodegradability of the organic contaminants
- Environmental factors which affect microbial activity
- Site hydrogeology.

Bioreclamation can be expected to reduce the concentration of only those organic compounds which are amenable to biological degradation. These are compounds that are either substrates for microbial growth and metabolism (the organism uses the compound as a carbon and energy source), or are cometabolically broken down as the microorganism uses another primary substrate as its carbon and energy source.

An extensive review of research on the relative biodegradabilities of environmental pollutants can be found in Evaluation of Systems to Accelerate Stabilization of Waste Piles (USEPA, 1985). Relative aerobic biodegradability of compounds can also be estimated using laboratory data associated with oxygen requirements for decomposition [i.e., 5-day and 21-day biological oxygen demand (BOD_5, BOD_{21}), chemical oxygen demand (COD), and the ultimate oxygen demand (UOD).] Table 9-1 presents relative biodegradabilities by adapted sludge cultures of various substances in terms of a BOD_5/COD ratio. A higher BOD_5/COD ratio represents a higher relative biodegradability. Chemical Property Estimation Methods (Lyman, Reehl, and Rosenblatt, 1982) provides additional information on methods of estimating biodegradability.

Table 9-2 summarizes organic groups subject to microbial metabolism by aerobic respiration, anaerobic respiration, and fermentation. "Oxidation" indicates that the compound is used as a primary substrate, and "co-oxidation" indicates that the compound is cometabolized. These tables and estimation methods provide only a general indication of degradability of compounds. In most instances, treatability studies will be required to determine degradability of specific waste components.

For most compounds, the most rapid and complete degradation occurs aerobically. There are some compounds, most notably the lower molecular weight halogenated hydrocarbons, which will only degrade anaerobically. [However, recent research conducted at the USEPA Robert S. Kerr Laboratory has discovered degradation of TCE in the presence of oxygen and methane gas (Wilson, 1984)].

It can be generalized that for the degradation of petroleum hydrocarbons, aromatics, halogenated aromatics, polyaromatic hydrocarbons, phenols, halophenols, biphenyls, organophosphates, and most pesticides and herbicides, aerobic bioreclamation techniques are most suitable. For the degradation of halogenated lower molecular weight hydrocarbons, such as unsaturated alkyl

TABLE 9-1.
BOD_5/COD RATIOS FOR VARIOUS ORGANIC COMPOUNDS

Compound	Ratio
Relatively Undegradable	
Butane	~ 0
Butylene	~ 0
Carbon tetrachloride	~ 0
Chloroform	~ 0
1,4-Dioxane	~ 0
Ethane	~ 0
Heptane	~ 0
Hexane	~ 0
Isobutane	~ 0
Isobutylene	~ 0
Liquefied natural gas	~ 0
Liquefied petroleum gas	~ 0
Methane	~ 0
Methyl bromide	~ 0
Methyl chloride	~ 0
Monochlorodifluoromethane	~ 0
Nitrobenzene	~ 0
Propane	~ 0
Propylene	~ 0
Propylene oxide	~ 0
Tetrachloroethylene	~ 0
Tetrahydronaphthalene	~ 0
1-Pentene	< 0.002
Ethylene dichloride	0.002
1-Octene	> 0.003
Morpholine	≤ 0.004
Ethylenediaminetetracetic acid	0.005
Triethanolamine	< 0.006
o-Xylene	< 0.008
m-Xylene	< 0.008
Ethylbenzene	< 0.009
Moderately Degradable	
Ethyl ether	0.012
Sodium alkylbenzenesulfonates	~ 0.017
Monoisopropanolamine	≤ 0.02
Gas oil (cracked)	~ 0.02
Gasolines (various)	~ 0.02
Moderately Degradable (cont'd.)	
Mineral spirits	~ 0.02
Cyclohexanol	0.03
Acrylonitrile	0.031
Nonanol	> 0.033
Undecanol	≤ 0.04
Methylethylpyridine	0.04-0.75
1-Hexene	< 0.044
Methyl isobutyl ketone	≤ 0.044
Diethanolamine	≤ 0.049
Formic acid	0.05
Styrene	> 0.06
Heptanol	≤ 0.07
sec-Butyl acetate	0.07-0.23
n-Butyl acetate	0.07-0.24
Methyl alcohol	0.07-0.73
Acetonitrile	0.079
Ethylene glycol	0.081
Ethylene glycol monoethyl ether	< 0.09
Sodium cyanide	≤ 0.09
Linear alcohols (12-15 carbons)	> 0.09
Allyl alcohol	0.091
Dodecanol	0.097
Relatively Degradable	
Valeraldehyde	≤ 0.10
n-Decyl alcohol	> 0.10
p-Xylene	< 0.11
Urea	0.11
Toluene	< 0.12
Potassium cyanide	0.12
Isopropyl acetate	≤ 0.13
Amyl acetate	0.13-0.34
Chlorobenzene	0.15
Jet fuels (various)	~ 0.15
Kerosene	~ 0.15
Range oil	~ 0.15
Glycerine	≤ 0.16
Adiponitrile	0.17
Relatively Degradable (cont'd.)	
Furfural	0.17-0.46
2-Ethyl-3-propylacrolein	< 0.19
Methylethylpyridine	< 0.20
Vinyl acetate	< 0.20
Diethylene glycol monomethyl ether	≤ 0.20
Naphthalene (molten)	≤ 0.20
Dibutyl phthalate	0.20
Hexanol	~ 0.20
Soybean oil	~ 0.20
Paraformaldehyde	0.20
n-Propyl alcohol	0.20-0.63
Methyl methacrylate	< 0.24
Acrylic acid	0.26
Sodium alkyl sulfates	~ 0.30
Triethylene glycol	0.31
Acetic acid	0.31-0.37
Acetic anhydride	≥ 0.32
Ethylenediamine	≤ 0.35
Formaldehyde solution	0.35
Ethyl acetate	≤ 0.36
Octanol	0.37
Sorbitol	≤ 0.38
Benzene	< 0.39
n-Butyl alcohol	0.42-0.74
Propionaldehyde	< 0.43
n-Butyraldehyde	≤ 0.43
Relatively Degradable (cont'd.)	
Ethyleneimine	0.46
Monoethanolamine	0.46
Pyridine	0.46-0.58
Dimethylformamide	0.48
Dextrose solution	0.50
Corn syrup	~ 0.50
Maleic anhydride	≥ 0.51
Propionic acid	0.52
Acetone	0.55
Aniline	0.56
Isopropyl alcohol	0.56
n-Amyl alcohol	0.57
Isoamyl alcohol	0.57
Cresols	0.57-0.68
Crotonaldehyde	< 0.58
Phthalic anhydride	0.58
Benzaldehyde	0.62
Isobutyl alcohol	0.63
2,4-Dichlorophenol	0.78
Tallow	~ 0.80
Phenol	0.81
Benzoic acid	0.84
Carbolic acid	0.84
Methyl ethyl ketone	0.88
Benzoyl chloride	0.94
Hydrazine	1.0
Oxalic acid	1.1

Source: Lyman, Reehl and Rosenblatt, 1982

TABLE 9-2.
SUMMARY OF ORGANIC GROUPS SUBJECT TO BIODEGRADATION

Substrate Compounds	Respiration		Fermentation	Oxidation	Co-oxidation
	Aerobic	Anaerobic			
Straight Chain Alkanes	+	+	+	+	+
Branched Alkanes	+	+	+	+	+
Saturated Aklyl Halides		+		+	+
Unsaturated Alkyl Halides		+		+	
Esters, Glycols, Epoxides	+	+	+	+	
Alcohols	+	+		+	
Aldehydes, Ketones	+	+		+	
Carboxylic Acids	+	+		+	
Amides	+	+			
Esters	+	+			
Nitriles	+	+			
Amines	+	+			
Phthalate Esters	+	+		+	
Nitrosamines		+			

(continued)

TABLE 9-2. (continued)

Substrate Compounds	Respiration		Fermentation	Oxidation	Co-oxidation
	Aerobic	Anaerobic			
Thiols					
Cyclic Alkanes	+		+	+	+
Unhalogenated Aromatics	+	+		+	
Halogenated Aromatics	+	+		+	+
Simple Aromatic Nitro Compounds	+	+			
Aromatic Nitro Compounds With Other Functional Groups	+	+			+
Phenols	+	+	+	+	+
Halogenated Side Chain Aromatics	+		+	+	
Fused Ring Hydroxy Compounds	+				
Nitrophenols		+			
Halophenols	+			+	

(continued)

TABLE 9-2. (continued)

Substrate Compounds	Respiration		Fermentation	Oxidation	Co-oxidation
	Aerobic	Anaerobic			
Phenols - Dihydrides, Polyhydrides	+			+	+
Two & Three Ring Fused Polycyclic Hydrocarbons	+			+	
Biphenyls	+				
Chlorinated Biphenyls	+				
Four Ring Fused Polycyclic Hydrocarbons	+				
Five Ring Fused Polycyclic Hydrocarbons	+				
Fused Polycyclic Hydrocarbons	+				
Organophosphates	+	+			
Pesticides and Herbicides	+	+			+

.rce: Envirosphere, 1985

halides like PCE and TCE, and saturated alkyl halides like 1,1,1-trichloroethane and trihalomethane, anaerobic degradation under very reducing conditions appears to be the most feasible approach in terms of our current understanding of microbial degradative capability. However, aerobic degradation in the presence of methane gas appears promising for some low molecular-weight halogenated hydrocarbons.

The availability of the compound to the organism will also dictate its biodegradability. Compounds with greater aqueous solubilities are generally more available to degradative enzymes. For example, cis-1,2-dichloroethylene is preferentially degraded relative to trans-1,2- dichloroethylene. The most likely explanation for this is because "cis" is more polar than "trans" and is therefore more water soluble (Parsons et al., 1982). The use of surfactants can increase the solubility and therefore the degradability of compounds (Ellis and Payne, 1984).

Environmental factors which affect microbial activity and population size will determine the rate and extent of biodegradation. Hydrogeology will affect not only microbial activity, but also the feasibility of in-situ treatment. These factors include:

- Appropriate levels of organic and inorganic nutrients trace elements
- Oxygen concentration
- Redox potential
- pH
- Degree of water saturation
- Hydraulic conductivity of the soil
- Osmotic potential (including total dissolved solids)
- Temperature
- Competition, including the presence of toxins and growth inhibitors
- Predators
- Types and concentrations of contaminants.

Microorganisms, like all living organisms, require specific inorganic nutrients (e.g., nitrogen, phosphate-phosphorus, trace metals), and a carbon and energy source to survive. Many organic contaminants provide the carbon and energy and thus serve as primary substrates. If the organic compound which is the target of the bioreclamation is only degraded cometabolically, a primary substrate must be available. Aerobes need oxygen, nitrate respirers need nitrate, and sulfate respirers need sulfate. Various anaerobic populations require specific reducing conditions. Optimum microbial activity for bioreclamation purposes occurs within a pH range of 6.0 to 8.0, with slightly alkaline conditions being more favorable.

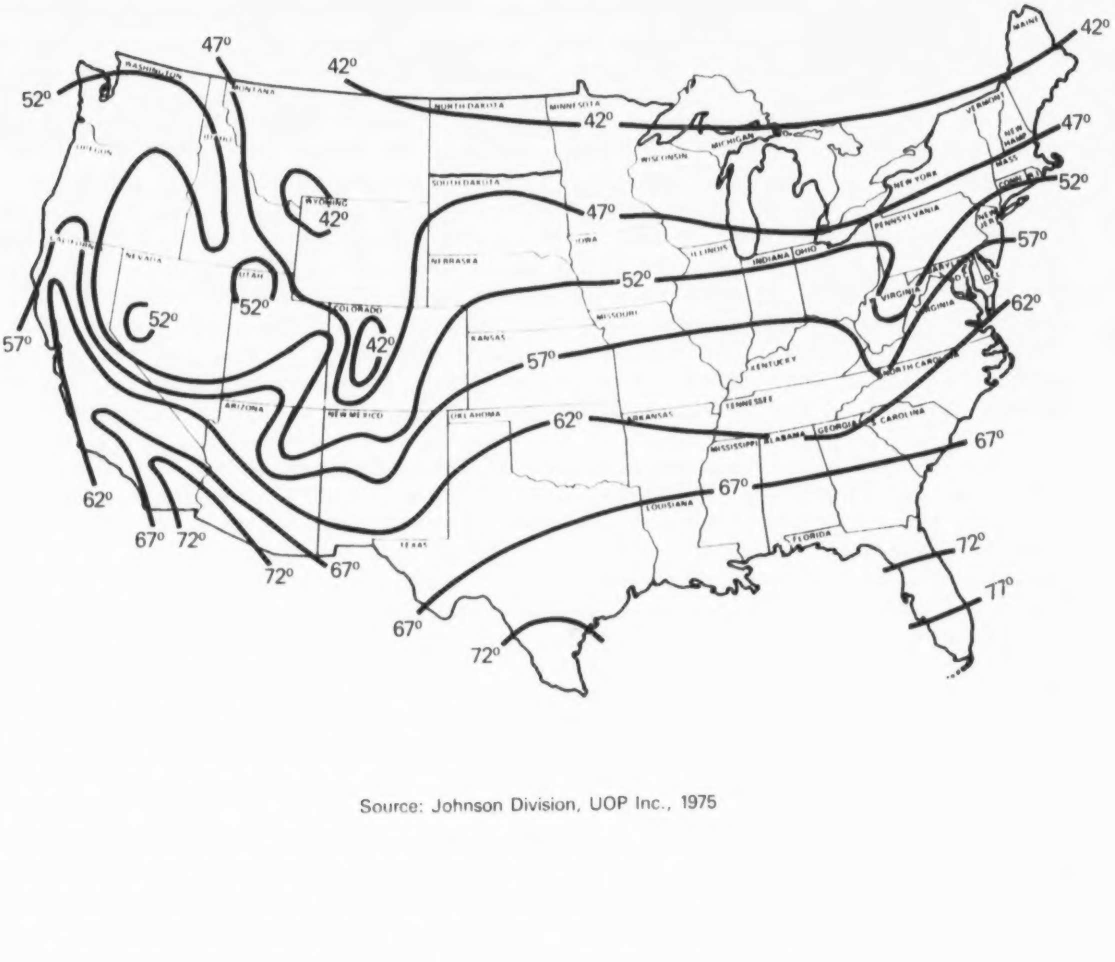
The temperature range for optimal organism growth in aerobic biological wastewater treatment processes has been found to range from 20° to 37°C (68° to 99°F). According to the "Q-10" rule, for every 10°C decrease in temperature in a specific system, enzyme activity is halved. Figure 9-1 illustrates typical groundwater temperatures throughout the United States. Although microbial populations in colder waters are adapted to lower temperatures, biodegradation rates can be expected to be much slower than at higher temperatures. It may not be feasible to attempt a bioreclamation approach in the extreme north.

Concentrations of inorganic and/or organic contaminants could be so high as to be toxic to the microbial populations. Table 9-3 lists concentrations at which certain compounds have been found to be toxic in industrial waste treatment. Microorganisms present in the subsurface may be more tolerant to high concentrations of these compounds. This determination must be made on a case-by-case basis. Conversely, a situation may prevail in which the contaminant concentrations are so low (<0.1 mg/l) that the assimilative processes of the microorganisms are sometimes not stimulated, thus adaptation to the particular substrate will not occur and the substrate will not be degraded (SCS Engineers, 1979). It is also possible that even if the contaminant is present in acceptable concentrations, if there is another "preferred" carbon source available, the microorganisms will catabolize it preferentially.

It is feasible to manipulate some of these factors in-situ to optimize environmental conditions. Nutrients and oxygen (or NO^-_3) can be added to the subsurface. It may be feasible in some cases to enhance reducing conditions, thereby lowering the redox potential. The pH can be adjusted with the addition of dilute acids or bases. Water could be pumped into an arid zone. Bioreclamation could be preceded by other treatments which could reduce toxic concentrations to a tolerable level. Even raising the temperature of a contaminated zone by pumping in heated water or recirculating groundwater through a surface heating unit may be feasible under conditions of low groundwater flow. This was done at a bioreclamation site in West Germany to increase the groundwater temperature 10°C (See Table 9-4) (Stief, 1984).

There are some factors that cannot be corrected, such as the presence of predators, competition between microbial populations, or the salinity of groundwater. This points to one of the advantages of relying on indigenous microorganisms rather than added microorganisms to degrade wastes. Although the added specialized microorganisms may have a superior degradation capability as developed in the laboratory or enriched in a surface biological reactor, they may not be able to survive subsurface conditions (e.g., salinity, light intensity, temperature, type of predators). However, through countless generations of evolution, natural populations have developed which are ideally suited for survival and proliferation in that environment. This is particularly true of uncontrolled hazardous waste sites where microorganisms have been exposed to the wastes for years or even decades. However, use of specialized microorganisms can be expected to have the greatest

FIGURE 9-1.
TYPICAL GROUNDWATER TEMPERATURES (°F) AT 100 FT. DEPTH IN THE UNITED STATES



Source: Johnson Division, UOP Inc., 1975

TABLE 9-3.
PROBLEM CONCENTRATIONS OF SELECTED CHEMICALS

Chemical	Problem Concentration (mg/l)	
	Substrate(1)	Non-Substrate(2)
n-Butanol	--	>1000
sec-Butanol	--	>1000
t-Butanol	>1000	>1000
Allyl alcohol	--	>1000
2-Ethyl-1-hexanol	500-1000	--
Formaldehyde	--	50-100
Crotonaldehyde	200	50-100
Acrolein	--	--
Acetone	--	>1000
Methyl isobutyl ketone	>1000	>1000
Isophorone	>1000	--
Diethylamine	--	300-1000
Ethylenediamine	--	100-300
Acrylonitrile	>1000	100
2-Methyl-5-ethylpyridine	>1000	100
N,N-dimethylaniline	>1000	--
phenol	>1000	300-1000
Ethyl benzene	>1000	--
Ethyl acrylate	600-1000	300-600
Sodium acrylate	--	>500
Dodecane	>1000	--
Dextrose	>1000	>1000
Ethyl acetate	>1000	--
Ethylene glycol	>1000	>900
Diethylene glycol	--	>1000
Tetraline	>1000	--
Karosene	--	>500
Cobalt chloride	--	>1000

(1)Substrate limiting reprsents the condition in which the subject compound is the sole carbon and energy source.

(2)Non-substrate limiting represents the condition in which other carbon and energy sources are present.

Source: SCS Engineers, 1979

TABLE 9-4.
SITE RECLAMATIONS USING BIODEGRADATION, INDIGENOUS MICROORGANISMS

Location	Contaminant	Waste Site Characteristics	Treatment	Comments	Reference
Ambler, PA 1972	Gasoline spill	Soil and groundwater contamination in a dolomite aquifer	Physical recovery followed by in-situ treatment. Nutrient solution delivered; air sparged through wells. Producing wells controlled groundwater flow.	No gasoline left in aquifer 10 months after treatment; estimated from 744 to 944 barrels degraded	Raymond, Jamison, and Hudson, 1976
Millville, NJ 1976	Gasoline spill	Soil and groundwater contamination in a sandy aquifer	Physical recovery followed by in-situ treatment. Nutrient solution delivered; air sparged through wells. Producing wells controlled groundwater flow.	No free product detected at end of program; phenol concentrations reduced to acceptable levels	Raymond et al., 1978
LaGrange, OR 1982	Gasoline leak	Soil and groundwater contamination in a shallow, highly permeable aquifer	Physical recovery followed by in-situ treatment. Nutrient addition; groundwater recycled through site. Air supplied at bottom of injection trench with diffusers.	System operated one year after which no free product detected	Minugh et al., 1983
Biocraft Labs Waldwick, NJ 1981	Methylene chloride, butanol, dimethyl aniline, acetone	Soil and groundwater contamination in a layer of glacial till	Physical recovery followed by surface biological treatment with indigenous microorganism; reinjection and in-situ treatment using aeration wells. Producing wells controlled groundwater flow.	Concentrations reduced from 100 to 700 mg/l down to 0.1 mg/l or less in groundwater after two year program	Jhaveri and Mazzacca, 1984
Karlsruhe, West Germany 1982	Petroleum products, cyanide	Soil and groundwater contamination in a sandy/gravelly aquifer	Groundwater pumped to surface, treated with ozone, then reinfiltrated.	Drinking water quality produced	Nagel et al., 1982 in Lee and Ward, 1984
Frankenthal, West Germany	Aromatic and aliphatic hydrocarbons including benzene, styrene, toluene, xylene, and napthalene	Soil and groundwater contamination in a sandy, high permeability aquifer	Physical recovery followed by in-situ treatment. Nitrate respiration enhanced by the addition of nitrate. Recirculated flushing water stripped and filtered before reinjection. Water temperature increased 10°C.	In 3 months aromatics gone and aliphatics reduced to 1/3 initial concentrations	Stief, 1984

(Continued)

TABLE 9-4. (continued)

Location	Contaminant	Waste Site Characteristics	Treatment	Comments	Reference
Granger, IN 1984	Gasoline leak	Soil and groundwater contamination in a sandy aquifer	Physical recovery followed by in-situ treatment. Hydrogen peroxide as oxygen source and nutrients added in-line to recycled water.	Program evaluation not yet available	API, Washington, DC, personal communication
Kelly Air Force Base, TX 1985	Jet fuel hydrocarbons, halogenated alkanes and aromatics, heavy metals	Hazardous waste disposal site; soil and groundwater contamination in clayey, low permeability aquifer	In-situ treatment planned. Nutrients and hydrogen peroxide will be added to recycled groundwater.	Treatment to commence Spring 1985	Wetzel, Henry, and Spooner, 1985

application at spill sites where the exposure time has not been long enough for a substantial adapted indigenous population to evolve.

Significant and active microbial populations have been found in the subsurface. Many types of bacteria have been isolated from subsurface soils and groundwater, and a considerable amount of research has recently been conducted to enumerate and characterize subsurface populations (e.g., Hirsch and Rades-Rohkohl, 1983; White et al., 1983; Ghiorse and Balkwill, 1983; Wilson et al., 1983; Ehrlich et al., 1983, Ventullo and Larson, 1983; and Harvey, Smith, and George, 1984).

Research confirms that substantial adapted populations do exist in contaminated zones and that bacterial numbers are elevated in contaminated zones relative to uncontaminated zones, i.e., the organic contaminants are being metabolized, leading to an increase in bacterial biomass. For example, in one study, significant bacterial populations were found in groundwater contaminated with gasoline, fuel oil, and other petroleum products (Litchfield and Clark, 1973). Groundwater containing less than 10 ppm of these hydrocarbons generally had populations of less than 10^3 organisms/ml, whereas groundwater with greater than 10 ppm hydrocarbons contained populations on the order of 10^6 organisms/ml. An investigation of a hazardous waste site contaminated with high concentrations of jet fuel hydrocarbons, industrial solvents, and heavy metals revealed bacterial numbers on the order of 10^7 cells/wet gram sample in core samples taken from the upper unsaturated zone and the saturated zone (Wetzel, Henry, and Spooner, 1985). A biodegradation study conducted with subsurface soil and groundwater from this site revealed that both aerobic and anaerobic populations were present that were capable of degrading the organic contaminants.

Even if substantial, active microbial populations are present, the wastes are biodegradable, and there are parameters that can be altered to optimize biodegradation in-situ, bioreclamation will not be feasible if the hydrogeology of the site is not suitable. The hydraulic conductivity must be great enough and the residence time short enough so that added substances, oxygen, and nutrients for example, are not "used up" before reaching the distal portions of the treatment zone. Sandy and other highly permeable sites will be far easier to treat than sites containing clayey soils.

There is also the possibility that added substances may react with the soil components. Oxidizing the subsurface could result in the precipitation of iron and manganese oxides and hydroxides. If this is extensive, the delivery system and possibly even the aquifer could become clogged. The in-situ bioreclamation of a site near Granger, Indiana developed problems with precipitation and clogging of the aquifer (see Table 9-4). Addition of phosphates could result in the precipitation of calcium and iron phosphates. If calcium concentrations are high, the added phosphate can be tied up by the calcium, and would therefore not be available to the microorganisms.

Heavy metals, bound in the soil matrix, could be mobilized into the groundwater. In reduced conditions, especially when there is ample organic carbon available, metals are likely to be bound in the soil as organic/metal chelates and as sulfides. When oxidized, the metal cations could coprecipitate with ferric hydroxide and/or precipitate in calcium/phosphate complexes. If iron and phosphate precipitation does not occur to a significant extent, the soluble metallic cations will remain in the aqueous phase. The soils may also plug as a result of excess biological growth.

9.1.3 Design Considerations

Biological treatment at contaminated sites encompasses both in-situ treatment approaches and treatment approaches involving groundwater withdrawal and treatment in biological reactors on the surface. This section addresses in-situ treatment and the combined use of in-situ and aboveground treatment.

9.1.3.1 Aerobic Bioreclamation

The first site remediation to treat hydrocarbon contamination in-situ was conducted by Raymond, Jamison, and coworkers at Suntech in the early 1970s. The first treatment approaches involved stimulating the indigenous microflora through the delivery of nutrients and air to the subsurface. Considerable developments have been made since the early 1970s, and many different treatment approaches have been used successfully to enhance biodegradation in contaminated zones. The indigenous microflora have been used in some site cleanups to degrade wastes. Specialized microorganisms, either adapted strains or genetically altered strains, have been used at other site remediations. Air was used in the earlier site remediations to provide oxygen. Hydrogen peroxide or possibly ozone now appear to be feasible alternatives to air or pure oxygen as a oxygen source. The earlier applications involved gasoline spills (Raymond, Jamison, and Hudson, 1976). Biological degradation is now being tested at a hazardous waste disposal site which contains a complex range of organics (Wetzel, Henry, and Spooner, 1985). Table 9-4 lists site reclamations that have involved stimulating the indigenous microflora. Table 9-5 lists site reclamations in which specialized microorganisms were used.

a. Oxygen Supply

Oxygen can be provided to the subsurface through the use of air, pure oxygen, hydrogen peroxide, or possibly ozone. Table 9-6 summarizes the advantages and disadvantages of the oxygen supply alternatives.

Air can be added to extracted groundwater before reinjection, or it can be injected directly into the aquifer. The first method, in-line aeration, involves adding air into the pipeline and mixing it with a static mixer, for example (Figure 9-2). This can provide a maximum of approximately 10 mg/l O_2.

TABLE 9-5.
EXAMPLES OF BIOLOGICAL RENOVATION AT CONTAMINATED SITES

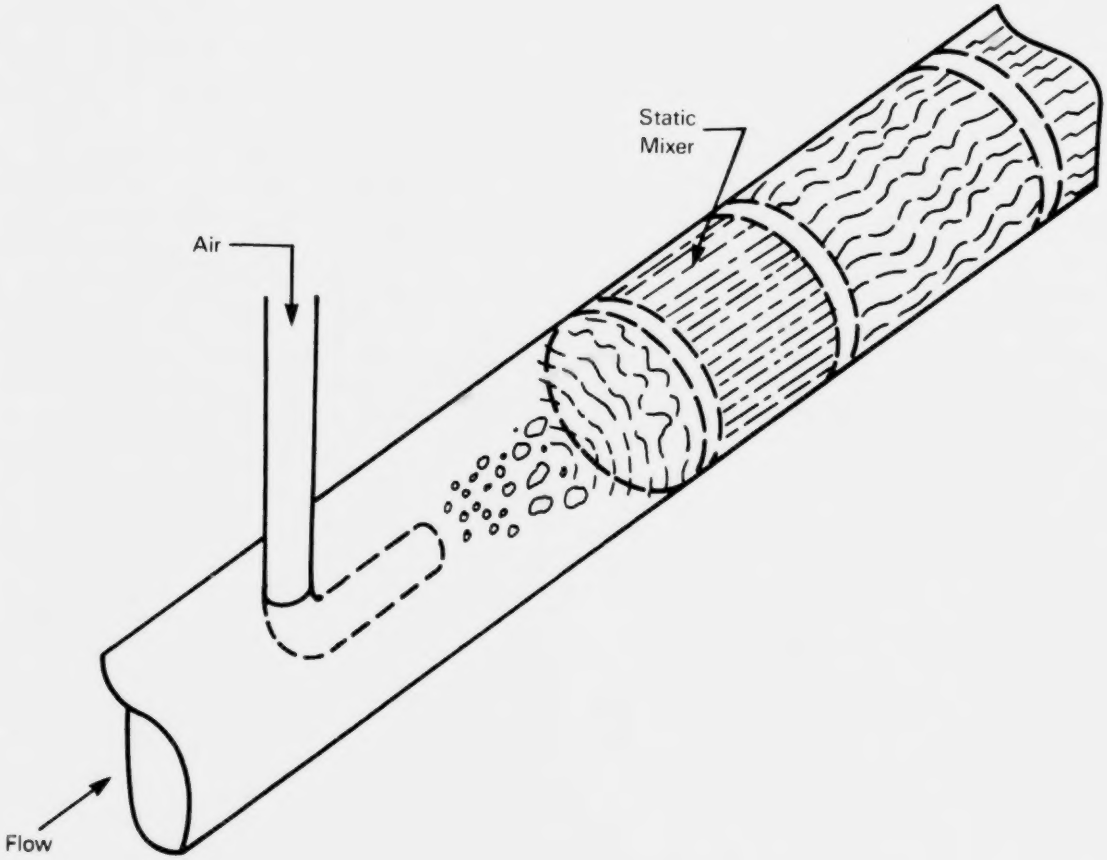
Waste Site Contaminant	Concentration ppm	Waste Site Characteristics	Biological Agent	Treatment Time (Days)	Residual Concentration ppm
Styrene	25	Railroad tankcar spill, area soils contaminated to a depth of 8 inches	BI-CHEM SUS-8	21	less than 1
Atrazine	Saturated Soil	50 Acre field	BI-CHEM-PBO-6		
Sludge containing Tricholorophenate	300	Sludge spread on soil to a depth of 6 inches	BI-CHEM-GEC-1	28	less than 1
Petroleum Distillate	12,000	Spill covering 4 acres at an oil tank farm	BI-CHEM-SUS-8	21	less than 1
Acrylonitrile	1,000	Soil and groundwater Contamination	PHENOBAC	90	less than 1
Formaldehyde	1,400	Soil and groundwater contamination	PHENOBAC	22	less than 1
Ortho-Chloro-phenol	15,000	Soil and groundwater Contamination	MUTANT BACTERIA	274	less than 1

Source: USEPA, 1982a

TABLE 9-6.
OXYGEN SUPPLY ALTERNATIVES

Substance	Application Method	Advantages	Disadvantages
Air	In-line	• Most economical	• Not practical except for trace contamination < 10 mg/l COD
	In-situ wells	• Constant supply of oxygen possible	• Wells subject to blow out
Oxygen-Enriched Air or Pure Oxygen	In-line	• Provides considerably higher O_2 solubility than does aeration	• Not practical except for low levels of contamination < 25 mg/l COD
	In-situ wells	• Constant supply of oxygen possible	• Very expensive • Wells subject to blow out
Hydrogen Peroxide	In-line	• Moderate cost • Intimate mixing with groundwater • Greater O_2 concentrations can be supplied to the subsurface (100 mg/l) H_2O_2 provides 50 mg/l O_2) • Helps to keep wells free of heavy biogrowth	• H_2O_2 decomposes rapidly upon contact with soil, and oxygen may bubble out prematurely unless properly stabilized
Ozone	In-line	• Chemical oxidation will occur, rendering compounds more biodegradable	• Ozone generation is expensive • Toxic to microorganisms except at low concentrations • May require additional aeration

FIGURE 9-2
CONFIGURATION OF STATIC MIXER



Source: USEPA, 1985

This concentration is sufficient only for degradation of about 5 mg/l hydrocarbons, and would therefore provide an inadequate oxygen supply. A pressurized line can increase oxygen concentrations as can the use of pure oxygen.

The equilibrium oxygen concentration in water increases with increased air pressure according to Henry's Law (Sawyer and McCarty, 1967):

where: $C_L = PH_k$

C_L = concentration of oxygen in liquid (mg/l)

= volume fraction (0.21 for O_2 in air)

P = air pressure (atm)

H_k = Henry's Law Constant for oxygen.

The value of Henry's Law Constant is 43.8 mg/l-atmosphere at 68°F (20°C). Pressure increases with groundwater depth at the rate of 0.0294 atmospheres per foot.

The use of in-situ aeration wells (Figure 9-3) is a more suitable method for injecting air into contaminated leachate plumes. A bank of aeration wells can be installed to provide a zone of continuous aeration through which the contaminated groundwater would flow. Oxygen saturation conditions can be maintained for degrading organics during the residence time of groundwater flow through the aerated zone. The required time for aeration can be derived from bench-scale studies. Residence time (t_r) through the aerated zone can be calculated from Darcy's equation (Freeze and Cherry, 1979) using groundwater elevations (i.e., head) and hydraulic conductivity as follows:

$$t_r = (L_a)^2/K(h_1-h_2)$$

where:

t_r = residence time (sec)

K = hydraulic conductivity (ft/sec)

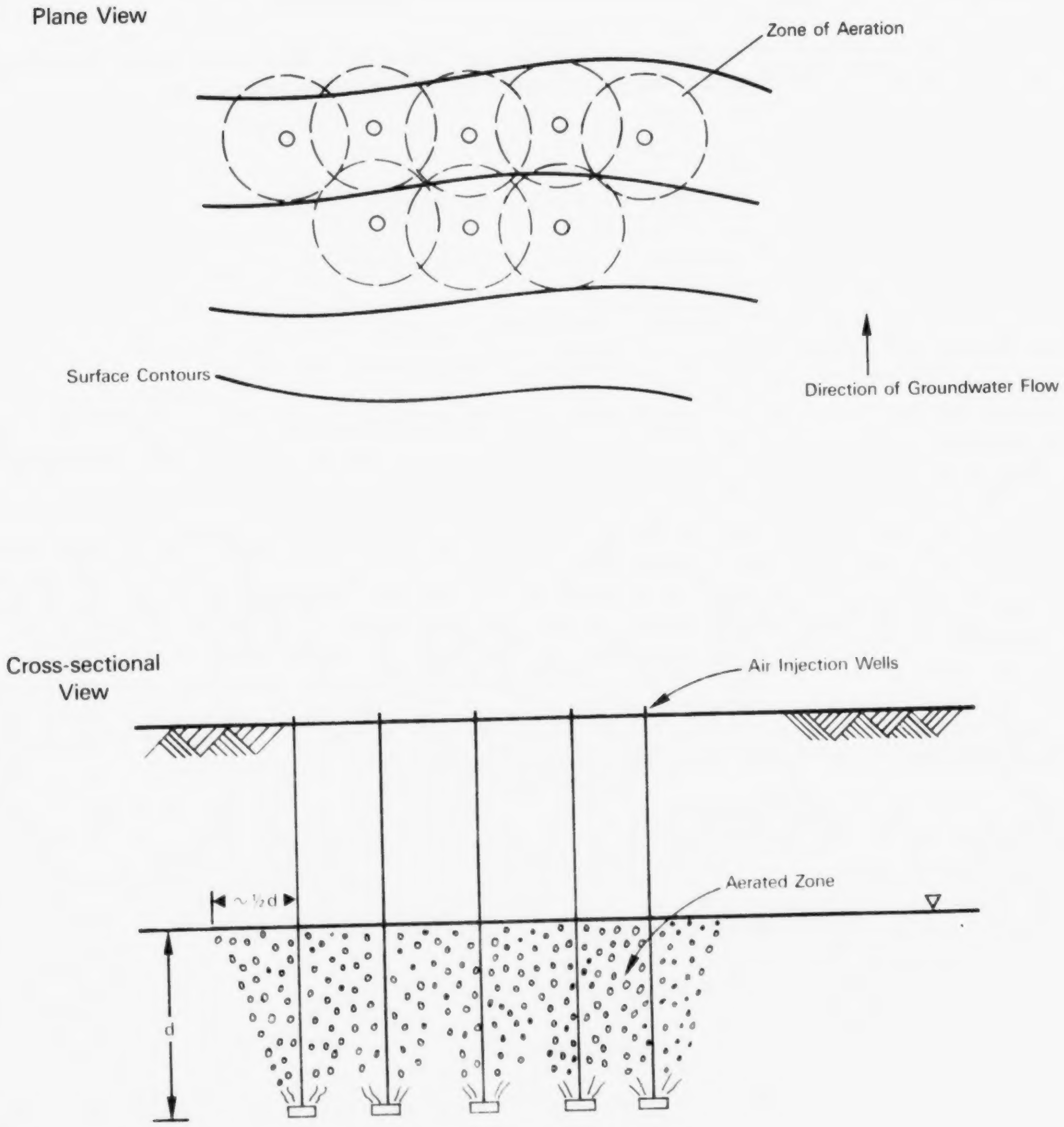
L_a = length of aerated zone (ft)

h_1 = groundwater elevation at beginning of aerated zone (ft)

h_2 = groundwater elevation at end of aerated zone (ft).

In the design of an in-situ aeration well zone system, the zone must be wide enough to allow the total plume to pass through. The flow of air must be sufficient to give a substantial radius of aeration while small enough to not cause an air barrier to the flow of groundwater.

Figure 9-3
POSSIBLE CONFIGURATION OF IN SITU AERATION WELL BANK



Source: USEPA, 1985

Various methods can be used to inject air or pure oxygen. Air has been sparged into wells using diffusers. For example, Raymond et al. (1975), sparged air into wells using diffusers attached to paint sprayer type compressors which could deliver approximately 2.5 cubic feet per minute. They were fitted with steel end plates and fittings to accommodate a polyethylene air line and a nylon rope and were suspended into the wells.

A blower can also be used to provide the flow rate and pressure for aeration. At the groundwater bioreclamation project in Waldwick, NJ, 5 pounds per square inch pressure is maintained in nine 10-foot aeration wells, each with an air flow of 5 cubic feet per minute (Groundwater Decontamination Systems, Inc., 1983).

Microdispersion of air in water using colloidal gas aprons (CGA) creates bubbles 25 to 50 micrometers in diameter. This is a newly developed method which holds great promise as a means of introducing oxygen to the subsurface (Michelsen, Wallis, and Sebba, 1984). With selected surfactants, dispersions of CGA's can be generated containing 65 percent air by volume.

Oxygenation systems, either in-line or in-situ can also be installed in order to supply oxygen to the bioreclamation process. Their advantage over conventional aeration is that higher oxygen solubilities and hence, more efficient oxygen transfer to the microorganisms can be attained. Solubilities of oxygen in various liquids are four to five times higher under pure oxygen systems than with conventional aeration. Therefore, in-line injection of pure oxygen will provide sufficient dissolved oxygen to degrade 20 to 30 mg/l of organic material, assuming 50 percent cell conversion. The higher oxygen solubilities may provide some flexibility in the design of cell banks, especially at greater pressures, since the oxygen may not be used up immediately, as with aeration.

Hydrogen peroxide (H_2O_2) as an oxygen source has been used successfully at the cleanup of several spill sites (Brubaker, G.R. FMC Aquifer Remediation Systems, Princeton, NJ, personal communication, 1985). Advantages of hydrogen peroxide include:

- Greater oxygen concentrations can be delivered to the subsurface. 100 mg/l H_2O_2 provides 50 mg/l O_2.
- Less equipment is required to oxygenate the subsurface. Hydrogen peroxide can be added in-line along with the nutrient solution. Aeration wells are not necessary.
- Hydrogen peroxide keeps the well free of heavy biogrowth. Microbial growth and subsequent clogging is sometimes a problem in air injection systems (Yaniga, Smith, and Raymond, 1984).

Hydrogen peroxide is cytotoxic, but research has demonstrated that it can be added to acclimated cultures at up to 1,000 ppm without toxic effects (Texas Research Institute, 1982). The remediation at Granger, Indiana, involved adding an initial concentration of 100 ppm, and increasing it to 500 ppm over the course of the treatment (API, Washington, DC, personal communication, 1985).

Hydrogen peroxide decomposes to oxygen and water ($H_2O_2 \quad O_2 + H_2O$). In the subsurface, hydrogen peroxide decomposition is catalyzed by chemical and biological factors. There has been some concern that decomposition could occur so rapidly that oxygen would bubble out near the site of injection and no oxygen would be made available to the distal portions of the treatment zone. Research has shown that high concentrations of phosphates (10 mg/l) can stabilize peroxide for prolonged periods of time in the presence of ferric chloride, an aggressive catalyst (Texas Research Institute, 1982). However, there are problems associated with adding such high phosphate concentrations to the subsurface, such as precipitation. One company claims to have developed specially stabilized hydrogen peroxide products for aquifer remediation. However, process performance information on these products is not available.

Ozone is used for disinfection and chemical oxidation of organics in water and wastewater treatment. In commercially available ozone-from-air generators, ozone is produced at a concentration of one to two percent in air (Nezgod, W. PCI Ozone Corporation, West Caldwell, NJ, personal communication, 1983). In bioreclamation, this ozone-in-air mixture could be contacted with pumped leachate using in-line injection and static mixing or using a bubble contact tank. A dosage of 1 to 3 mg/l of ozone can be used to attain chemical oxidation (Nezgod, W. PCI Ozone Corporation, West Caldwell, NJ, personal communication, 1983). However, German research on ozone pretreatment of contaminated drinking waters indicates that the maximum ozone dosage should not be greater than 1 mg/l of ozone per mg/l total organic carbon; higher concentrations may cause deleterious effects to microorganisms (Rice, R.G. Rip G. Rice, Inc., Ashton, MD, personal communication, 1983). At many sites, this may limit the use of ozone as a pretreatment method to oxidize refractory organics, making them more amenable to biological oxidation.

A petroleum products spill in Karlsruhle, Germany, was cleaned up in-situ using ozone as an oxygen source for biological degradation (Nagel et al., 1982 in Lee and Ward, 1984). The groundwater was pumped out, treated with ozone, and recirculated. Approximately one gram of ozone per gram of dissolved organic carbon was added to the groundwater and was allowed a contact time of four minutes in the aboveground reactor. This increased the oxygen content to 9 mg/l with a residual of 0.1 to 0.2 gram of ozone per cubic meter in the treated water.

b. Nutrients

Nitrogen and phosphate are the nutrients most frequently present in limiting concentrations in soils. Other nutrients required for microbial metabolism include potassium, magnesium, calcium, sulfur, sodium, manganese, iron, and trace metals. Many of these nutrients may already be present in the aquifer in sufficient quantities and need not be supplemented.

The optimum nutrient mix can be determined by laboratory growth studies and from geochemical evaluations of the site. Caution must be exercised in evaluating microbial needs based on soil and groundwater chemical analysis. Chemical analysis does not necessarily indicate what is available to the microorganisms. In some cases generalizations can be made, e.g., if calcium is present at 200 mg/l (a very high concentration), it is likely that calcium supplementation is unnecessary.

The form of nutrients may or may not be critical in terms of microbial requirements, depending on the site. Studies have shown that forms of nitrogen and phosphate were not critical for microorganisms (Jamison, Raymond, and Hudson, 1976). However, it has been recommended than an ammonia-nitrogen source is preferable to a nitrate-nitrogen source because ammonia-nitrogen is more easily assimilated by microorganisms (FMC, 1985). Nitrate is also a pollutant limited to 10 mg/l in drinking water.

The site geochemistry may be a critical factor in determining the form of nutrients, as well as the added concentrations. For example, use of diammonium phosphate could result in excessive precipitation (Jamison, Raymond, and Hudson, 1976) and nutrient solution containing sodium could cause dispersion of the clays, thereby reducing permeability (Anderson, D., K.W. Brown, and Associates, Inc., College Station, TX, personal communication, 1985). Where calcium is high, it is likely to lead to the precipitation of added phosphate, rendering it unavailable to microbial metabolism. If a site is likely to encounter problems with precipitation, iron and manganese addition may not be desirable. If the total dissolved solids content in the water is extremely high, it may be desirable to add as little extra salts as possible.

The compositions of some basal salts media are given in Tables 9-7 and 9-8. Only phosphate and nitrogen had to be added to a site in Ambler, PA. Bulk quantities of ammonium sulfate [$(NH_4)_2SO_4$)], disodium phosphate (Na_2HPO_4), and monosodium phosphate (NaH_2PO_4) were mixed in a 2,200 gallon tank truck and added to the groundwater in the form of a 30 percent concentrate in water which was metered into the injection wells (Raymond, Jamison, and Hudson, 1976). Phosphate concentrations in injection wells varied from 200 to 5,800 mg/l throughout the site cleanup. Phosphate concentrations in all wells were determined weekly and injection rates were adjusted accordingly.

GDS, Inc. used the basal salt medium listed in Table 9-8 in the combined surface/in-situ treatment system at the Biocraft site (Jhaveri and Mazzacca, 1984). The nutrient solution used at the Granger, Indiana, site was composed of ammonium nitrate and disodium phosphate (FMC, 1985).

An organic carbon source, such as citrate or glucose, could be added if the compound of interest is only degraded cometabolically and a primary carbon source is required. Such additions could also be made when low levels of contaminants are present and are not sufficient to sustain an active microbial population. Citrate, or another chelate such as EDTA, could be added to hold metals in solution if water is alkaline, a condition under which metals may

TABLE 9-7
COMPOSITION OF BASAL SALTS MEDIUM

Salt Type	Concentration (mg/l)
KH_2PO_4	400
Na_2HPO_4	600
$(NH_4)NO_3$	10
$MgSO_4 \cdot 7H_2O$	200
Na_2CO_3	100
$CaCl_2 \cdot 2H_2O$	10
$MnSO_4 \cdot H_2O$	20
$FeSO_4 \cdot 7H_2O$	5

Source: Jamison, Raymond, and Hudson, 1976

TABLE 9-8
BASAL SALT MEDIUM USED BY GDS INC.

Salt	Concentration (mg/l)
NH_4Cl	500
KH_2PO_4	270
K_2HPO_4	410
$MgSO_4$	1.4
Na_2CO_3	9
$CaCl_2$	0.9
$MnSO_4$	1.8
$FeSO_4$	0.45

Groundwater Decontamination Systems, Inc., 1983 precipitate. Citrate, however, will be preferentially degraded relative to other organics, and could slow the degradation of contaminants. Addition of low concentrations of a source of amino acids, such as peptone or yeast extract, could promote biodegradation. However, high concentrations of these compounds could inhibit degradation of contaminants because of preferential degradation.

c. Design of Delivery and Recovery Systems

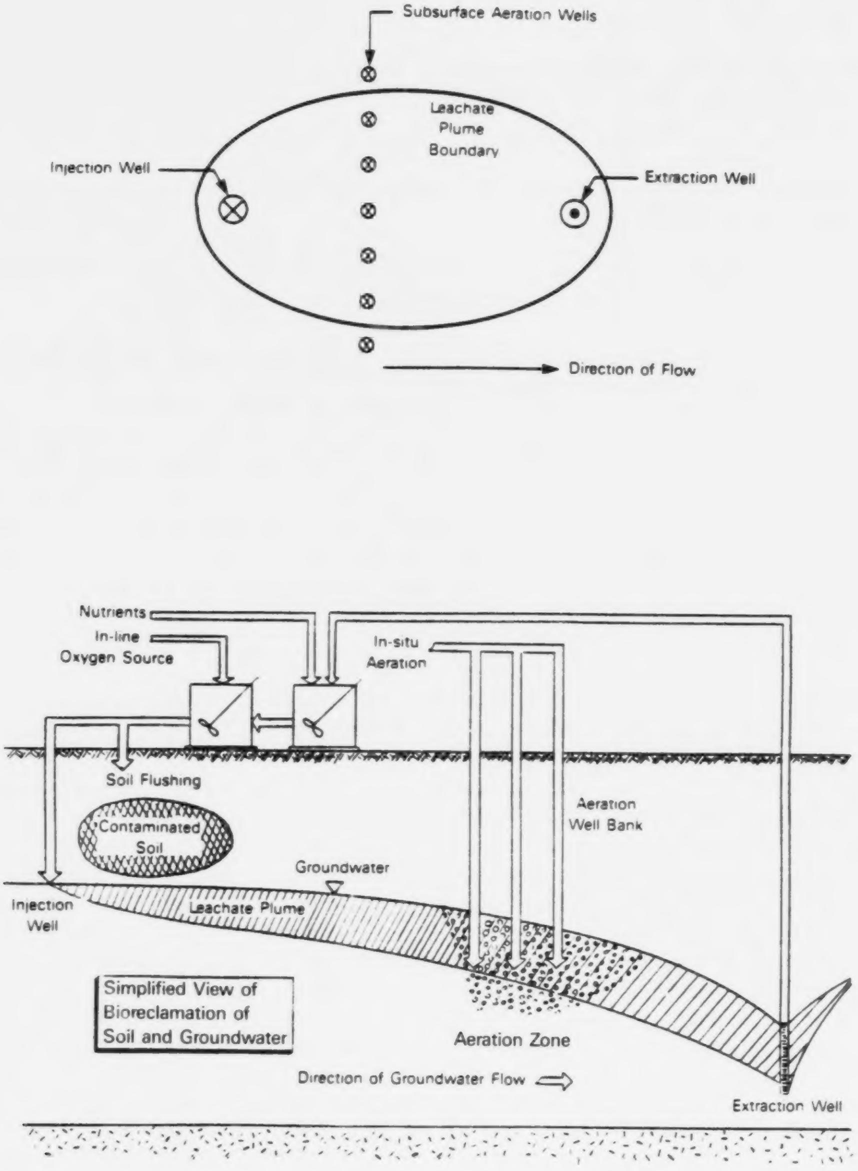
One of the major factors determining success of an in-situ treatment system is to ensure that the injection and recovery systems are designed to accomplish the following:

- Provide adequate contact between treatment agents and contaminated soil or groundwater
- Provide hydrologic control of treatment agents and contaminants to prevent their migration beyond the treatment area
- Provide for complete recovery of spent treatment solutions and/or contaminants where necessary.

A number of design alternatives are available for delivering nutrients and oxygen to the subsurface and for collecting and containing the groundwater. These methods can generally be categorized as gravity flow or forced methods. Most of the systems that have been used for bioreclamation have involved the use of subsurface drains (gravity system), injection wells and extraction wells. Subsurface drains and extraction wells are described in detail in Sections 5.2 and 5.1, respectively. Some examples of delivery and recovery systems are described below.

Figure 9-4 illustrates a hypothetical configuration in which groundwater is extracted downgradient of the zone of contamination and reinjected upgradient. In-situ aeration supplies oxygen directly to the contaminated plume while nutrients and oxygen are added in-line by way of mixing tanks. Treated water is infiltrated through contaminated soil in order to flush contaminants from the soil. Extraction and injection wells can be used to treat contaminants to almost any depth in both the saturated and unsaturated zone. However their use becomes cost-prohibitive in very low permeability soils because of the need to space the wells very close together to ensure complete delivery or recovery. Subsurface drains can be used under conditions of moderately low permeability although delivery and recovery of chemicals will be slow. They are generally limited to depths of 40 feet or less because of the cost associated with excavation (shoring, dewatering, hard rock excavation) of the trench. Surface gravity delivery systems (e.g., spray irrigation, flooding, ditches) which involve application of treatment solutions directly to the surface, as illustrated by "surface flushing" in Figure 9-4, are most effective for treating shallow contaminated zones located

FIGURE 9-4
SIMPLIFIED VIEW OF GROUNDWATER BIORECLAMATION



in the unsaturated zone. They can also be used to treat contaminants in the saturated zone, provided the following conditions are met:

- The soil above the saturated zone (through which treatment solutions percolate) is sufficiently permeable to allow percolation of treatment solutions to the groundwater within a reasonable length of time

- Groundwater flow rates must be sufficient to ensure complete mixing of the treatment solutions with the groundwater.

The feasibility and effectiveness of these methods is affected by topography and climate.

Figure 9-5 shows the design of a groundwater injection/recovery system which is currently being used for bioreclamation at an Air Force site. (SAIC/JRB, 1985). The system, which is designed to operate in moderately low permeability soils, consists of nine pumping wells and four injection wells. Groundwater is pumped at an even rate from the pumping wells to a central flow equalization (surge tank). Flow is metered from this tank into a length of pipe into which measured amounts of nutrients and hydrogen peroxide are added. The treated water then flows to a distribution box to be distributed at an even rate to each of the four injection wells. Overflow from the equalization tank will flow into an on-site storage tank.

The injection/recovery system was designed using a two-dimensional, geohydrologic non-steady flow model which simulated the flow of groundwater at the site in response to an injection/recovery pumping system.

Important criteria used for the design of the injection/recovery system include the following:

- The groundwater injection rate will be the same as the rate determined during the field testing program

- All injected groundwater and associated elements are to be kept within the site boundary to prevent the transport of contaminants to adjacent areas (this implies that there may be some net groundwater pumpage at the site)

- The distance between the injection-pumping wells should be such that approximately six injection-pumping cycles can be completed within a 6-month period.

Figure 9-6 illustrates an injection trench used in the treatment of the Biocraft site (Jhaveri and Mazzacca, 1983). The trench was 10 feet deep by 4 feet wide by 100 feet long. The trench had a 15 mil plastic liner installed on the bottom, back, ends, and top such that reinjected water only flowed out of the front (downgradient) face of the trench. About 40 feet of slotted steel pipe was installed horizontally in the trench to carry reinjected water into the trench system. As water flowed into the injection trench, the water was forced to exit only from the front face. Backflow is minimized by this

FIGURE 9-5. PLAN VIEW OF EXTRACTION/INJECTION SYSTEM USED AT AN AIR FORCE SITE

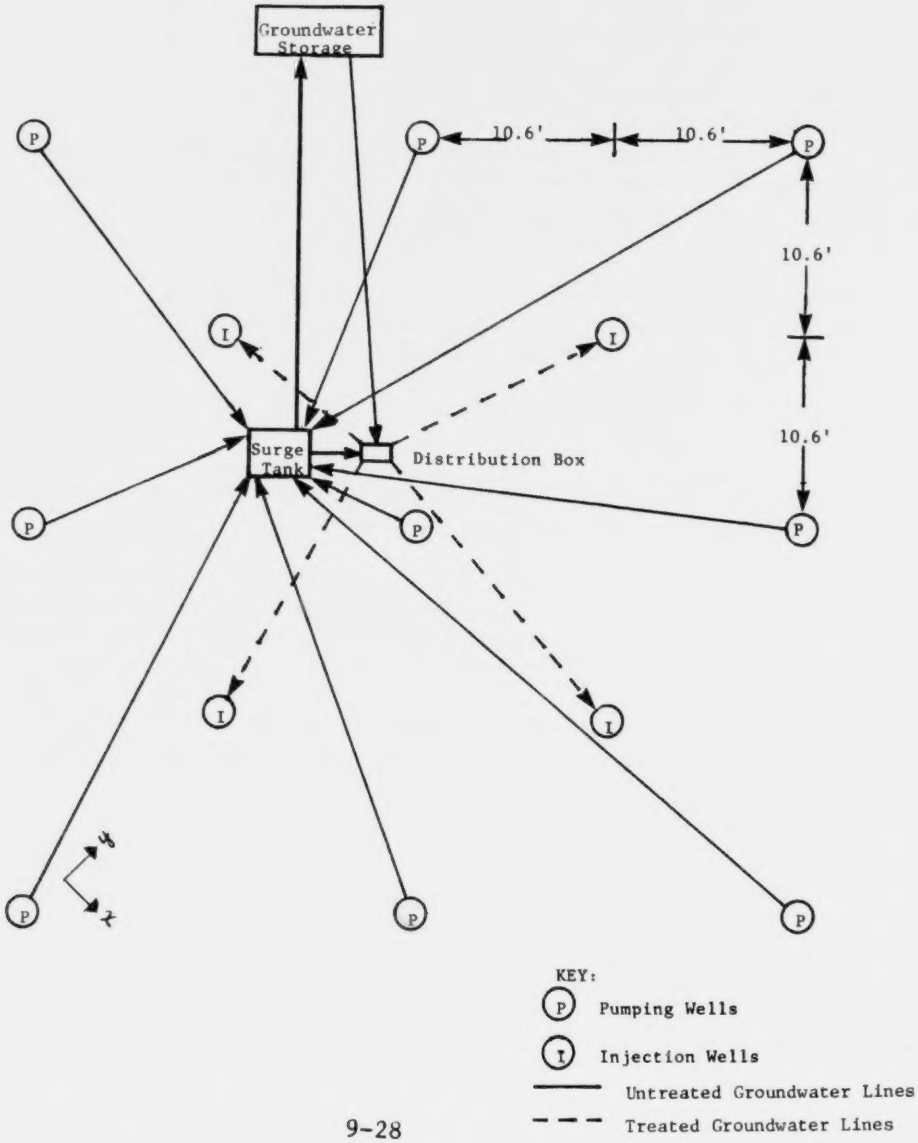
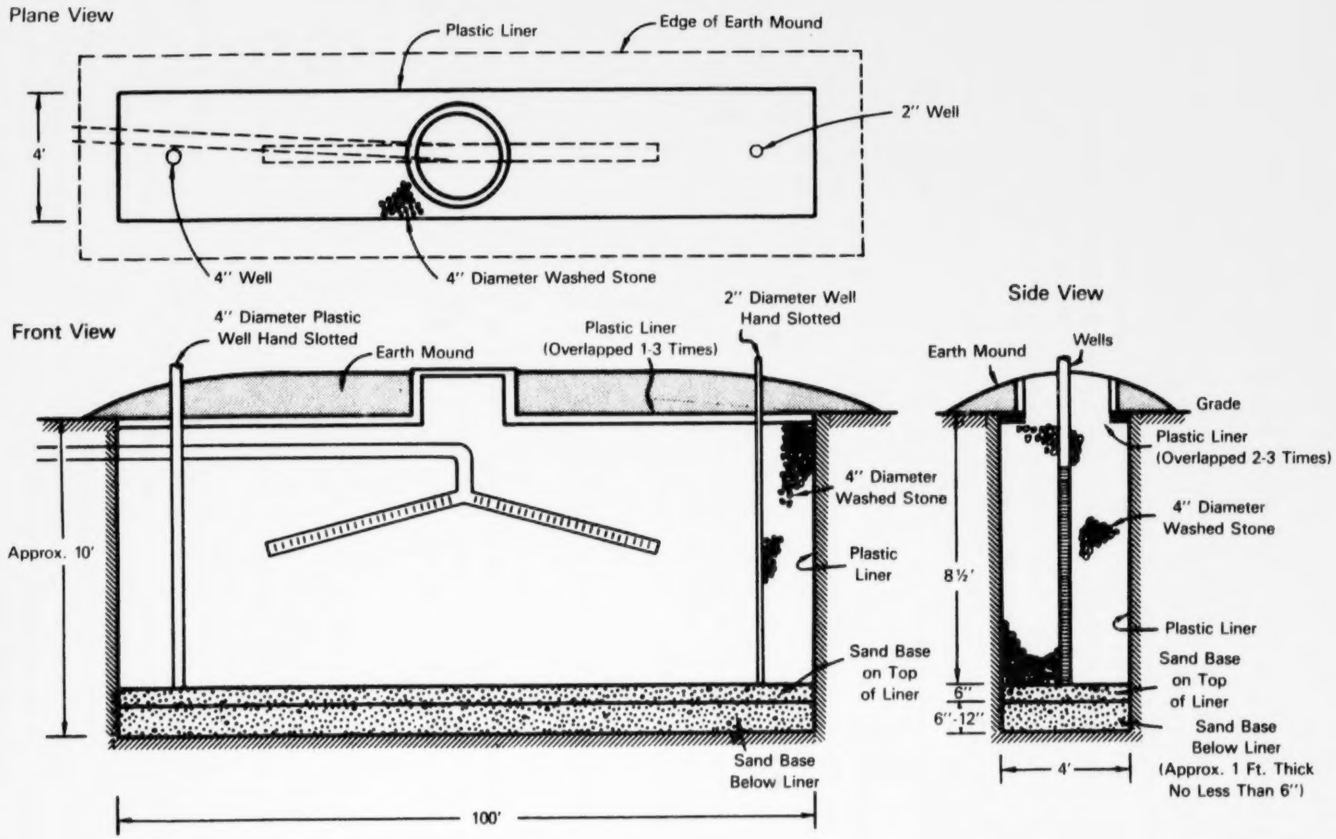


FIGURE 9-6. CONFIGURATION OF REINJECTON TRENCHES


Plane View
Plastic Liner
Edge of Earth Mound
4'
2'' Well
4'' Well
4'' Diameter Washed Stone
Front View
4'' Diameter Plastic Well Hand Slotted
Earth Mound
Plastic Liner (Overlapped 1-3 Times)
2'' Diameter Well Hand Slotted
4'' Diameter Washed Stone
Plastic Liner
Sand Base on Top of Liner
Sand Base Below Liner
Approx. 10'
100'
Side View
Earth Mound
Wells
Grade
Plastic Liner (Overlapped 2-3 Times)
4'' Diameter Washed Stone
Plastic Liner
Sand Base on Top of Liner
Sand Base Below Liner (Approx. 1 Ft. Thick No Less Than 6'')
8½'
6''
6''-12''
4'

Note: Treated water exits only from one side of the trench.

Source: Jhaveri and Mazzacca, 1983

design feature. Barriers can also be used behind the trench and extended to a point where backflow is further minimized. In extreme cases, total control over backflow and plume containment can be obtained by installing a circumferential wall barrier.

Optimum extraction and injection flow rates will many times be predetermined by aquifer yield limits or hydraulic design for plume containment. The factors affecting aquifer flow rates are described in Section 5.1.

Aquifer flow rates should be sufficiently high so that the aquifer is flushed several times over the period of operation. Thus, if the cleanup occurs over a three year period, flow rates between injection and extraction wells should be such that a residence time of one-half year or less occurs between the well pairs. This corresponds to six or more flushes. Several recycles would cause flushing of soils containing organics, preventing the clogging caused by microorganism buildup because of increased flow rate; more even distribution of nutrients and organic concentration within the plume; and better and more controlled degradation. Flow rates and recycle should not be high enough to cause excessive pumping costs, loss of hydraulic containment efficiency because of turbulent conditions, corrosion, excessive manganese deposition, flooding, or well blow out. The operating period will depend on the biodegradation rate of the contaminants in the plume and the amount of recycle. If the period of operation is excessively long, for example more than five years, the operating costs of bioreclamation may outweigh the capital costs of another remedial alternative.

9.1.3.2 Anaerobic Bioreclamation

Anaerobic treatment is generally not as promising for site remediation as aerobic treatment. Anaerobic processes are slower, fewer compounds can be degraded, and the logistics of rendering a site anaerobic have not been developed to date.

Anaerobic metabolism includes: (1) anaerobic respiration, in which nitrate or sulfate may be used by nitrate or sulfate reducers as a terminal electron acceptor, and (2) interactive fermentative/methanogenic processes, which are carried out by what is referred to as a methanogenic consortium.

If it were possible to provide proper reducing conditions, degradation by methanogenic processes would be promising. A considerable body of research indicates that methanogenic consortiums are active in the subsurface and are capable of degrading certain organics (Ehrlich et al., 1982; Parsons et al., 1982; and Suflita and Gibson, 1984). Most notably, methanogenic consortiums are able to degrade TCE, PCE, and other lower molecular weight halogenated organics which generally cannot be degraded by aerobic or other respiratory processes. Reductive dehalogenation appears to be the primary mechanism involved in degradation. Methanogenic consortiums are also able to degrade various aromatics, halogenated aromatics, and some pesticides. Degradation of petroleum hydrocarbons, straight-chain and branched alkanes and alkenes, is not possible under methanogenic conditions.

Methanogenic activity requires a very low redox potential, -250 mv or less. No O_2, NO_3^-, or SO_4^{-2} can be present or the redox potential will not be low enough. Currently, there are no demonstrated methods for rendering a site anaerobic. When the contamination is shallow, there is an aquitard below the zone of contamination, and the flow of groundwater can be contained, it might be possible to induce reducing conditions by flooding the site, as one would a rice paddy. Another possible method or rendering the site anaerobic would be to add excessive amounts of easily biodegradable organics so that the oxygen would be depleted. One other promising possibility might be to circulate groundwater to the surface through anaerobic digesters or anaerobic lagoons. These methods may require long retention times because of slow degradation rates under anaerobic conditions. There have been no reports of pilot or field studies using anaerobic degradation under methanogenic conditions.

Nitrate respiration may be a feasible approach to decontaminating an aquifer. Denitrification (the reduction of NO_3 to NH_3 or N_2) has been demonstrated to occur in contaminated aquifers. Nitrate respiration was used successfully in the treatment of an aquifer contaminated with aromatic and aliphatic hydrocarbons (see Table 9-4) (Stief, 1984). Nitrate can be added in-line along with other nutrients and intimate mixing with groundwater can occur. The cost is moderate; all that is required is the nutrient feed system and an in-line mixer.

Nitrate, however, is a pollutant, limited to 10 ppm in drinking water. Consequently, it may be more difficult to obtain permits for use of nitrate at a site than for oxygen or hydrogen peroxide. Additionally, degradation rates under aerobic conditions are more rapid and a broader range of compounds can be degraded. There is no reason why nitrate respiration would be a better treatment approach given the amount of success that has been demonstrated with aerobic treatment approaches to date.

9.1.4 Operation and Maintenance

Operation and maintenance of a bioreclamation process involve aspects of the hydraulic system as well as the biological system. The hydraulic aspects relate to pumps, extraction and injection wells, and injection trenches; these are discussed in Sections 5.1 and 5.2.

Monitoring a number of parameters is necessary to determine process performance. Monitoring of groundwater can be performed at the injection and extraction wells, as well as at monitoring wells. Monitoring wells should be placed on-site to monitor process performance and off-site to monitor for pollutant migration as well as to provide background information on changes in subsurface conditions due to seasonal fluctuation. Table 9-9 lists parameters which should be monitored, and suggests methods which can be used to monitor these parameters.

TABLE 9-9.
RECOMMENDED PARAMETERS TO MONITOR

Parameter	Location of Analysis	Media	Analytical Method
Total Organic Carbon (TOC)	laboratory	groundwater	TOC analyzer
Priority pollutant analysis or analysis of specific organics	laboratory	soil and groundwater	GC/MS(1)
Microbiology-cell enumerations	laboratory	soil and groundwater	Direct counts. Plate counts on groundwater media or enriched media.
	field	groundwater	Plate counts with portable water test kits (e.g. Soil Test Inc., Evanston, IL).
Temperature conductivity dissolved oxygen (DO) pH	field	groundwater	In-situ water quality monitoring instrument or prepackaged chemicals, field test kits.
Alkalinity Acidity, M&P Chloride Hardness (total NH_3-N NO_3-N PO_4, all forms SO_4 TDS (total dissolved solids)	field	groundwater	Prepackaged chemicals/ field test kits; water analyzer photometer (Soil Test. Inc., Evanston, IL; Lamotte Chemical, Chestertown, MD).
Heavy metals (if present)	field laboratory	groundwater soil and groundwater	Prepackaged chemicals/ test kits; GC/MS; AAS.(2)

(continued)

TABLE 9-9. (continued)

Parameter	Location of Analysis	Media	Analytical Method
Hydrogen peroxide (H_2O_2)	field	groundwater	Prepackaged chemicals for H_2O_2, test strips available. Titanium sulfate titration and spectrophotometer analysis for H_2O_2 for greater accuracy.

(1) GC/MS = gas chromatography/mess spectrometry

(2) AAS = Atomic absorption spectrometry

In a biological system, pH should be maintained in a range between 6 and 8 and concentrations of both nutrients and organics should be kept as uniform as possible to protect against shock loading. Dissolved oxygen should be maintained above the critical concentration for the promotion of aerobic activity, which ranges from 0.2 to 2.0 mg/l, with the most common being 0.5 mg/l (Hammer, 1975).

Clogging of the aquifer, injection wells or trenches, or extraction wells by microbiological sludge is a possibility. GDS Inc. installed two wells in each of their injection trenches in case flushing was ever required to remove sludge. After 1-1/2 years of operation, clogging had not occurred (Groundwater Decontamination Systems Inc., 1983). However, problems with biofouling and plugging of sparging points was encountered during a spill cleanup conducted by Groundwater Technology (Yaniga, Smith, and Raymond, 1984). This interfered with oxygen transfer and necessitated frequent mechanical cleaning. When hydrogen peroxide was substituted for air sparging in order to deliver increased quantities of oxygen to the aquifer, one added benefit was that the hydrogen peroxide kept the wells free of heavy biogrowth.

The permeability of the aquifer could be reduced due to precipitation, as discussed in Section 9.2.2. Other factors, such as dispersion of clays, could reduce aquifer permeability. When calcium concentrations are high in the soil, phosphates can be rapidly attenuated due to precipitation with calcium, becoming unavailable for microbial metabolism. Nutrient formulations should be devised with the help of experienced geochemists which will minimize problems with precipitation and dispersion of clays. One company claims to have developed special soil preconditioners and nutrient formulations which reduce these problems and maximize nutrient mobility and solubility; however, no process performance data are available on these products.

Maintenance of the bacterial population at their optimal levels is also important, especially for selective mutant organisms which tend to be more sensitive than naturally occurring species. A continuous incubation facility operating at higher temperatures and under more controlled conditions could be used to maintain the microbial population. The high biomass-containing stream formed from such a facility could then be reinjected via wells or trenches so as to reinoculate the subsurface continuously with microorganisms.

Aeration wells may be particularly susceptible to operational problems. If injected gas fluidizes the material around it, soil substrata shifts can occur which may cause a well blowout (free passage of air to the surface). The cone of influence in a blown out well will be greatly reduced, therefore requiring the installation of a new well. The best method to prevent blowouts is to keep gas velocities below those necessary to cause fluidization, or to place wells deep enough so that overburden pressure prevents excessive fluidization, or both (Sullivan, Chemineer Kinecs, Dayton, OH, personal communication, 1983). Suntech stated that a number of aeration wells became inoperative because of blowout during their groundwater cleanup in 1972 and had to be replaced (Raymond, Jamison, and Hudson, 1976). This suggests that aeration well blowout could become a commonly encountered problem if attention is not paid to the design criteria.

9.1.5 Technology Selection/Evaluation

Aerobic bioreclamation has been demonstrated to be effective in degrading organics at more than 30 spill sites. Although it has not yet been demonstrated at hazardous waste sites, it can be expected to be effective and reliable provided the organics are amenable to aerobic degradation and the hydraulic conductivity of the aquifer is sufficiently high. There are substantial research data to suggest that microorganisms found at uncontrolled hazardous waste sites are well-acclimated to the wastes. Effectiveness and reliability could be adversely affected by factors, such as precipitation, which could reduce the permeability of an aquifer.

Relative to conventional pump and treat methods, bioreclamation may be more effective since it is capable of degrading organics sorbed to soils. Sorbed organics are not removed using conventional pump and treat methods.

The nature of the delivery systems can effect the reliability of the bioreclamation approach. Pumping systems are prone to mechanical and electrical failure. However, repairs can be made relatively quickly. Subsurface drains are less prone to failure since there are no electrical components. Where mechanical failures do occur, repairs can be both costly and time consuming.

Implementation of a remedial action involving bioreclamation will take longer than excavation and removal of contaminated soils. Depending upon the site, it could also take longer than a conventional pump and treat approach. The advantage of in-situ bioreclamation over a pump and treat approach is that in-situ biodegradation treats contaminated subsurface soils, thereby removing the source of groundwater contamination.

The increased time required for in-situ bioreclamation is dependent primarily upon the degradation rates, which are in turn dependent upon oxygen availability. The form in which oxygen is delivered to the subsurface and the aquifer permeability are the critical factors in this respect. As discussed previously, far more oxygen can be delivered to the subsurface in the form of hydrogen peroxide.

Other aspects of implementation are similar to implementation of conventional pumping or subsurface drain systems with a few exceptions. Depending upon the hydraulic conductivity, drains or wells must generally be spaced closer together to ensure nutrient and oxygen availability at all portions of the zone being treated. The lower the flow rate of the nutrient/oxygen-enriched water and the more rapidly nutrients and oxygen are attenuated, the closer the injection wells or drains must be spaced. Well/drain spacing will also be dictated by the need, if any, to contain the contaminated plume or treatment solution.

There are few additional safety hazards associated with in-situ bioreclamation aside from those hazards normally associated with being on a hazardous waste site or a drill site. Since wastes are treated in the ground, the danger of exposure to contaminants is minimal during a bioreclamation operation relative to excavation and removal.

A nutrient/oxygen or nutrient/hydrogen peroxide solution does not represent an environmental threat. Most of the nutrients will be utilized and attenuated by microbial activity. If the form of the nutrient is carefully selected (e.g. ammonia-nitrogen rather than nitrate-nitrogen), the remaining nutrients will not present an environmental threat. The hydrogen peroxide will rapidly decompose in the subsurface to oxygen and water.

The only treatment reagent which could pose a hazard, if used, is the concentrated hydrogen peroxide solution prior to mixing with groundwater. Worker protection for operations involving hydrogen peroxide outside of a closed container or pipe should include the use of chemically resistant gloves, an apron, and a face shield. Safety training in the use of hydrogen peroxide should be provided by qualified personnel.

9.1.6 Costs

Costs for biological in-situ treatment are determined by the nature of the site geology and geohydrology, the extent of contamination, the kinds and concentrations of contaminants, and the amount of groundwater and soil requiring treatment. There is no easy formula for predicting costs. Costs provided for actual site cleanups indicate that biological treatment can be far more economical as an alternative to, or in conjunction with, excavation and removal or conventional pump and treat methods.

In-situ treatment costs include costs for well construction and pumping. These are provided in Sections 5.1 and 5.2. Unit costs for chemicals, nutrients, and hydrogen peroxide are provided in Table 9-10. Cost data for an actual and hypothetical site cleanups involving in-situ treatment are presented below.

Total capital and research and development costs for cleanup of the Biocraft site (Table 9-11) were $926,158 including $446,280 which were spent on process development (R&D). Project costs also included the hydrogeological study, and design and operation of the groundwater injection and collection system, and biostimulation plant. Total operating costs, based on treating 13,680 gallons/ day, were approximately $226/day, or $0.0165/gal. The total cost including amortization based on projected costs is $0.0358/gal over a three year period. Prior to the biological treatment program, contaminated water had been removed at a rate of 10,000 gal/month. The average disposal cost had been $0.35/gal (Jhaveri and Mazzacca, 1984). The cost of biological treatment of an equal number of gallons is an order of magnitude less than that for disposal. The Biocraft site employed surface biological reactors as well as enhancing in-situ treatment by reinfiltrating oxygen and nutrient treated groundwater. Costs for in-situ treatment alone would have been less because process plant design and equipment would not be included in an in-situ approach. (See Table 9-11).

Table 9-12 presents the estimated site cleanup costs for hypothetical sites involving the use of hydrogen peroxide as an oxygen source for the enhancement of in-situ biodegradation (FMC, 1985). The cleanup of 300 gallons of gasoline from a sand gravel aquifer over a period of 6 to 9 months is between $70,000 and $120,000 (Site A). Cleanup of 3,000 gallons of diesel fuel from a fractured bedrock formation is estimated to require 9 to 12 months and $160,000 to $250,000. The cost estimate for degrading 10,000 gallons of jet fuel from a fine gravel formation is estimated to cost $400,000 to $600,000 and take 14 to 18 months.

9.2 Chemical Treatment

9.2.1 General Description

Generally, organic and inorganic contaminants can be immobilized, mobilized for extraction, or detoxified. Technologies placed in the category "immobilization" include precipitation, chelation, and polymerization. The category encompassing methods for mobilizing contaminants for extraction is termed "soil flushing." Flushing agents include surfactants, dilute acids and

TABLE 9-10.
CHEMICAL COSTS

Category	Chemical	Cost/Unit
Acids	Hydrochloric acid, 20° Baume tanks	$55-105/ton
	Nitric acid 36°to 42° Baume tanks	$195/ton
	Sulfuric acid	
	virgin, 100%	$61-95.9/ton
	smelter, 100%	$48-65/ton
Bases	Caustic soda, liquid 50%, low iron	$255-285/ton
Chelating agents	Ammonium chloride	$18/100lbs
	Citric acid	$0.81-$1.19/lb
Fertilizers (Microbial nutrients)	Ammonia, anhydrous, fertilizer	$140-$215/ton
	Ammonium chloride	$18/100 lbs
	Ammonium sulfate	$73-79/ton
	Sodium monophosphate	$55.75/100 lbs
	Sodium diphosphate	$54.50/100 lbs
	Phosphoric acid	
	75%, commercial grade	$27.5/100 lbs
	52-54% a.p.a., agricultural grade	$3.10/unit-ton[a]
	Potassium muriate, 60 to 62%, minimum	$0.82-0.92/unit-ton
	Potassium chloride	$105/ton
	Potassium-magnesium sulfate	$59/ton
Liming material	Agricultural limestone (dolomite)	3.50-34/ton[b]
	Lime	$30.75-45/ton
	Hydrated lime	$32.5-34.5/ton
Oxidizing agents	Hydrogen peroxide, 35%	$0.24/lb
	Potassium permanganate	$1.03-1.06/lb
Reducing agents	Caustic soda, liquid 50%, low iron	$255-285/ton
Precipitating agents	Ferrous sulfate	
	heptahydrate	130/ton
	monohydrate	160/ton

(continued)

TABLE 9-10. (continued)

Category	Chemical	Cost/Unit
Surfactant		
Anionic	Witconate 605A	0.65-0.85/lb[c]
	Witconate P-1020BV (calcium sulfonates)	0.70-0.88/lb[c]
Nonionic	Adsee 799	0.75-0.87/b[c]

Source: Schnell, 1985, unless otherwise noted.

a. unit-ton: 1 percent of 2,000 pounds of the basic constituent or other standard of the material. The percentage figure of the basic constituent multiplied by the unit-ton price gives the price of 2,000 pounds of the material.
b. Source: USEPA, 1984a.
c. Source: Witco Chemical Corp., Houston, TX, personal communication, 1985: cost varies depending on quantity purchased (drum, truckload, or bulk).

bases, and water. Detoxification techniques include oxidation, reduction, neutralization, and hydrolysis.

These categories do not define the limits of each technology. For example, a treatment method that immobilizes a contaminant may also serve to detoxify it; a flushing solution that mobilizes one contaminant may precipitate, detoxify, or increase the toxicity of another.

Tables 9-13 and 9-14 provide a summary of those in-situ chemical treatment methods for organics and inorganics, respectively, that are most promising or have been most widely discussed in the literature. The compounds amenable to treatment, the treatment reagents, and the process are summarized.

9.2.2 Applications/Limitations

The feasibility of an in-situ treatment approach is dictated by site geology and hydrology, soil characteristics, and waste characteristics. Since the application of many chemical in-situ treatment techniques to hazardous waste disposal site reclamation is conceptual or in the developmental stage, there is little hard data available on the specific site characteristics that may limit the applicability of each method. A list of site and soil

TABLE 9-11.
SUMMARY OF PROJECT COSTS — BIOCRAFT LABORATORIES, WALDWICK, NJ

Task	Actual Expenditure	Unit Cost	Period of Performance
A. Hydrogeological Study-problem definition	$73,948	--	1976-1978
B. In-house Process Development (R&D)	$446,280	--	1978-1981
C. Ground Water collection/injection system total	$184,243	--	
1) Design	($61,490)		
2) Installation	($122,753)		1980-1981
D. Biostimulation plant Design and Construction total	$221,207	--	1981
1) Engineering Design	($58,400)	--	1981
2) Masonary Construction	($73,975)	--	1981
3) Equipment and Miscellaneous Installation	($88,832)	--	1981
CAPITAL AND R&D TOTAL	$926,158	--	
E. Operation & Maintenance (O&M)			
1) Utilities	$47.40 /day		
i) Electricity 26.4 kw(24 hours/day)	($46.82/day)	7.39¢/kwh	1983 rates
ii) Steam 72 pounds (33kg)/day @ 90PSI	(58¢/day)	0.8¢/pound	1981
2) Maintenance see text	$159.93/day		
3) Nutrient Salts see Table 15	$19.20/day	see Table 15	1983
Total Water treated - 13,680 gallons (51,779 l)/day	O. & M total:$226.53/ day	$0.0165/gallon ($0.0044/l)	

Source: USEPA, 1984b

TABLE 9-12.
ESTIMATED COSTS FOR HYPOTHETICAL BIORECLAMATIONS USING HYDROGEN PEROXIDE AS AN OXYGEN SOURCE

	Site A	Site B	Site C
Contaminant	300 gallons gasoline	2,000 gallons diesel fuel	10,000 gallons jet fuel
Formation	Sand/gravel	Fractured bed rock	Fine gravel
Flow Rate	50 gpm	10 gpm	100 gpm
Project Time	6-9 months	9-12 months	14-18 months
Estimated Costs	\$70-120M(1)	\$160-250M(1)	\$400-600M(1)

(1) M=1000

Source: FMC, 1985

characteristics considered important in evaluating the treatment applicability is provided in Table 9-15 (Sims and Wagner, 1983).

Most of the treatment approaches discussed in this section involve the delivery of a fluid to the subsurface. Therefore, the same factors that limit the use of injection/extraction wells, drains, or surface gravity application systems such as flooding and spray irrigation for bioreclamation will limit the applicability of most in-situ chemical treatment approaches. Minimal permeability requirements must be met if the treatment solution is to be delivered successfully to the contaminated zone. Sandy soils are far more amenable to in-situ treatment than clayey soils. Further, the contaminated groundwater must be contained within the treatment zone. Measures must be taken to ensure that treatment reagents do not migrate and, of themselves, become contaminants. Care must be taken during the extraction process not to increase the burden of contaminated water by drawing uncontaminated water into the treatment zone from the aquifer or from hydraulically connected surface waters.

Potential chemical reactions of the treatment reagents with the soils and wastes must be considered. Most hazardous waste disposal sites contain a mix of contaminants. A treatment approach that may neutralize one contaminant may render another more toxic or mobile; for example, chemical oxidation will

TABLE 9-13.
SUMMARY OF IN SITU CHEMICAL TREATMENT METHODS FOR ORGANICS

Method	Amenable to Treatment	Treatment reagents	Process
Soil Flushing			• Contaminated soils are flooded with water or a water chemical mixture and the elutriated solution is collected.
- Water flushing	Hydrophilic compounds (high solubility, low K_{ow})	Water	
- Water with surfactants	Hydrophobic compounds (low solubility, high K_{ow})	Aqueous solutions of surfactants	• Contaminants are mobilized into solution by reason of solubility, formation of emulsion or reaction.
Oxidation	Benzene and substituted benzenes Phenols Halogenated phenols Nitro aromatics PAHs Heterocyclic nitrogen and oxygen compounds Aldehydes and ketones Sulfides, disulfides	Ozone, hypochlorite, or hydrogen peroxide	• Oxidation state of compounds is increased by loss of electrons. • Contaminants are detoxified, mobility is increased or compounds are made more amenable to biological degradation.

(continued)

TABLE 9-13. (continued)

Method	Amenable to Treatment	Treatment reagents	Process
Hydrolysis (base-catalyzed)	Esters Amides Carbamates Organophosphorous compounds Certain pesticides (i.e., parathion, malathion, 2-4D Esters, DDT)	Water with lime or NaoH	• Attack of nucleophile (e.g., water or hydroxyl ion) on an electrophile (e.g., carbon or phosphorous) resulting in bond cleavage and displacement of the leaving group.
Polymerization	Aliphatic, aromatic and oxygenated monomers - vinyl chloride - isoprene - acrylonitrile	Catalyst activation	• Conversion of a compound to a larger chemical multiple of itself. • Reduces mobility of compound in soil.

TABLE 9-14
SUMMARY OF IN-SITU TREATMENT METHODS FOR INORGANICS

Method	Inorganics Amenable to Treatment	Treatment reagents	Process
Precipitation			Formation of insoluble metal precipitate thereby reducing the mobility of the metal
- sulfide	Heavy metals	Sodium or calcium sulfide	
- carbonate/ hydroxide	Heavy metals	Lime, calcium carbonate	
- phosphate	Heavy metals	Superphosphate fertilizer	
Soil Flushing			
- acids/bases	Heavy metals	Dilute solutions of acids or bases	Involves solubilizing the metals followed by extraction of the metal ions
- chelates	Heavy metals	Chelating agents such as citric acid or EDTA	Formation of stable metal chelates; depending on chelating agent, metal chelate is either strongly sorbed to soil or is highly mobile and can be flushed using water or dilute acid solutions.
Oxidation	Trivalent arsenic	Potassium permanganate	Oxidizes trivalent arsenic to pentavalent arsenic, and results in precipitation of arsenic-iron-manganese compounds.
Reduction	Hexavalent chromium Hexavalent selenium	Ferrous sulfate Ferrous sulfate	Reduces Cr (VI) to Cr (III) Reduces Se (VI) to Se (IV)

TABLE 9-15.
SITE AND SOIL CHARACTERISTICS IDENTIFIED AS IMPORTANT IN IN-SITU TREATMENT

Characteristics

- Site location/topography
- Slope of site-degree and aspect
- Soil, type and extent
- Soil profile properties
  - depth
  - boundary characteristics
  - texture*
  - amount and type of coarse fragments
  - structure*
  - color
  - degree of mottling
  - presence of carbonates
  - bulk density*
  - cation exchange capacity*
  - clay content
  - type of clay
  - pH*
  - Eh*
  - surface area*
  - organic matter content*
  - nutrient status*
  - microbial activity*
- Hydraulic properties and conditions
  - depth to impermeable layer or bedrock
  - depth to groundwater*, including seasonal variations
  - infiltration rates*
  - permeability* (under saturated and a range of unsaturated conditions)
  - water holding capacity*
  - soil water characteristic curve
  - field capacity/permanent wilting point
  - flooding frequency
  - run-off potential*
  - aeration status*

(continued)

TABLE 9-15. (continued)

Characteristics
Climatological factors
temperature*
wind velocity and direction

* Factors that may be managed to enhance soil treatment

Source: Sims and Wagner, 1983

(Manuscripts originally printed in the Proceedings of the National Conference on Management of Uncontrolled Hazardous Waste Sites. 1983. Available from Hazardous Materials Control Research Institute, 9300 Columbia Blvd., Silver Spring, MD 20910).

destroy or reduce the toxicity of many toxic organics, but chromium III, if present, will oxidize to the more toxic and mobile chromium VI state. The permeability of soils may be reduced by the treatment approach. In soils high in iron and manganese, for example, oxidizing the subsurface could result in the precipitation of iron and manganese oxides and hydroxides, which could clog the delivery system and the aquifer.

9.2.3 Soil Flushing

Organic and inorganic contaminants can be washed from contaminated soils by means of an extraction process termed "soil flushing," "solvent flushing," "ground leaching," or "solution mining." Water or an aqueous solution is injected into the area of contamination, and the contaminated elutriate is pumped to the surface for removal, recirculation, or on-site treatment and reinjection. During elutriation, sorbed contaminants are mobilized into solution by reason of solubility, formation of an emulsion, or by chemical reaction with the flushing solution.

Solutions with the greatest potential for use in soil flushing fall into the following classes:

- Water
- Acids-bases
- Complexing and chelating agents

- Surfactants
- Certain reducing agents.

Water can be used to flush water-soluble or water-mobile organics and inorganics. Hydrophilic organics are readily solubilized in water. Organics amenable to water flushing can be identified according to their soil/water partition coefficient, or estimated using the octanol/water coefficient. Octanol-water partition coefficients are available for a large number of compounds in: Substituent Constants for Correlation Analysis in Chemistry and Biology (Hansch and Leo, 1979). Chemical Property Estimation Methods (Lyman, Reehl and Rosenblatt, 1982) provides various methods for estimating the octanol-water partition coefficient using readily available physical and chemical data. Organics considered soluble in the environmental sense are ones with a partition coefficient (K) of approximately less than 1000 (log K = 3). High solubility organics, such as lower molecular weight alcohols, phenols, and carboxylic acids, and other organics with a coefficient less than 10 ($\log K \leq 1$) may have already been flushed from the site due to natural flushing processes. Medium solubility organics (K = 10 to 1000) which could be effectively removed from soils by water flushing include low to medium molecular weight ketones, Aldehydes, and aromatics, and lower molecular weight halogenated hydrocarbons such as trichloroethylene (TCE) and tetrachloroethylene (PCE). Inorganics which can be flushed from soil with water are soluble salts such as the carbonates of nickel, zinc, and copper. Adjusting the pH with dilute solutions of acids or bases (to be discussed) will enhance inorganic solubilization and removal.

The remedial actions at the Goose Farm site in Plumstead, NJ, involved the installation of a wellpoint collection and recharge system and the flushing of contaminants from the soil with water (USEPA, 1984b). Water flushing has been used over a 5 to 6 year period to reclaim a former herbicide factory site in Sweden (Truett, Holberger, and Sanning, 1982). Cleanup of a PCE spill in Germany involved water flushing of contaminated soils, and effected a 50 percent decrease in TCE over an 18 month period (Stief, 1984).

Dilute solutions of acids have been widely used in industrial processes to extract metal ions. Solutions of sulfuric, hydrochloric, nitric, phosphoric, and carbonic acid are used in industrial applications to dissolve basic metal salts (hydroxides, oxides, and carbonates). However, because of the toxicity of many acids, it is desireable to use weak acids for in-situ treatment. Sodium dihydrogen phosphate (NaH_2PO_4) and acetic acid (CH_3COOH) have low toxicity and are reasonably stable. A stronger dilute acid such as sulfuric acid would be used if the soil contained sufficient alkalinity to neutralize it. Acidic solutions may also serve to flush some basic organics such as amines, ethers, and anilines.

Complexing and chelating agents may also find use in a solution mining removal system for heavy metals. Some commonly employed substances are citric acid, ethylenediaminetetracetic acid (EDTA), and diethylenetriaminepentacetic acid (DTPA) (Rogoshewski and Carstea, 1980). Chelating agents used for in-situ treatment must result in a stable metal-chelate complex which is resistant to decomposition and degradation.

Another possibility for mobilizing metals which are strongly adsorbed to manganese and iron oxides in soils is to reduce the metal oxides which results in release of the heavy metal into solution. Chelating agents or acids can then be used to keep the metals in solution. Treatment agents which may be suitable for this purpose include hydroxylamine together with an acid, or sodium dithionite/citrate.

Surfactants can be used to improve the solvent property of the recharge water, emulsify nonsoluble organics, and enhance the removal of hydrophobic organics sorbed onto soil particles. Surfactants improve the effectiveness of contaminant removal by improving both the detergency of aqueous solutions and the efficiency by which organics may be transported by aqueous solutions (USEPA, 1985). Surfactant washing is among the most promising of the in-situ chemical treatment methods.

Numerous environmentally safe and relatively inexpensive surfactants are commercially available. Use of surfactants to date has been restricted to laboratory research. Most of the research has been performed by the petroleum industry for tertiary oil recovery (Barakat et al., 1983; Cash et al., 1977; Doe, Wade, and Schechter, 1977; and Wilson and Brandner, 1977). Aqueous surfactants have also been proposed for gasoline cleanup. In a study performed by the Texas Research Institute (1979) for the American Petroleum Institute, a mixture of an anionic and nonionic surfactants result in contaminant recovery of up to 40 percent. In a laboratory study conducted by Ellis and Payne (1983), crude oil recovery was increased from less than 1 percent to 86 percent, and PCB recovery was increased from less than 1 percent to 68 percent when soil columns were flushed with an aqueous surfactant solution.

Characteristics of surfactants and their environmental and chemical properties are listed in Table 9-16 (USEPA, 1985). This table can be used to aid in the preliminary selection of a surfactant. However, laboratory testing of the surfactant should be performed to verify surfactant properties.

An economically feasible soil flushing method may involve the recycling of elutriate through the contaminated material, with make-up solvent being added to the system while a fraction of the elutriate stream is routed to a portable wastewater treatment system. The appropriate types of wastewater treatment operations will depend on waste stream characteristics, and a discussion of their applications can be found in Section 10.1.

The advantages of the soil flushing process are that, if the waste is amenable to this technique and distribution, collection, and treatment costs are relatively low, solution mining can present an economical alternative to the excavation and treatment of the wastes.

TABLE 9-16.
SURFACTANT CHARACTERISTICS

Surfactant Type		Selected Properties and Uses	Solubility	Reactivity
ANIONIC	1) Carboxylic Acid Salts	• Good Detergency	• Generally Water Soluble	• Electrolyte Tolerant
	2) Sulfuric Acid Ester Salts	• Good Wetting Agents		• Electrolyte Sensitive
	3) Phosphoric & Polyphosphoric Acid Esters	• Strong Surface Tension Reducers	• Soluble in Polar Organics	• Resistant to Biodegradation
	4) Perfluorinated Anionics			• High Chemical Stability
	5) Sulfonic Acid Salts	• Good Oil In Water Emulsifiers		• Resistant to Acid and Alkaline Hydrolysis
CATIONIC				
	1) Long Chain Amines	• Emulsifying Agents	• Low or Varying Water Solubility	• Acid Stable
	2) Diamines & Polyamines	• Corrosion Inhibitor		
	3) Quaternary Ammonium Salts		• Water Soluble	• Surface Adsorption to Silicaeous Materials
	4) Polyoxyethylenated Long Chain Amines			
NON IONIC	1) Polyoxyethylenated Alkylphenols Alkylphenol Ethoxylates	• Emulsifying Agents	• Generally Water Soluble	• Good Chemical Stability
	2) Polyoxyethylenated Straight Chain Alcohols and Alcohol Ethoxylates	• Detergents	Water Insoluble Formulations	• Resistant to Biodegradation
	3) Polyoxyethylenated Polyoxypropylene Glycols	• Wetting Agents		• Relatively Non-Toxic
	4) Polyoxyethylenated Mercaptans	• Dispersents		
	5) Long-Chain Carboxylic Acid Esters	• Foam Control		• Subject to Acid and Alkaline Hydrolysis
	6) Alkylolamine "Condensates", Alkanolamides			
	7) Tertiary Acetylenic Glycols			

TABLE 9-16. (CONTINUED)

Surfactant Type	Selected Properties and Uses	Solubility	Reactivity
AMPHOTERICS			
1) pH Sensitive	• Solublizing Agents	• Varied (pH dependent)	• Non-Toxic
2) pH Insensitive	• Wetting Agents		• Electrolyte Tolerant
			• Adsorption to Negatively Charged Surfaces

Source: USEPA, 1985

9.2.4 Immobilization

Immobilization methods are designed to render contaminants insoluble and prevent leaching of the contaminants from the soil matrix and their movement from the area of contamination. Little is currently known about the effectiveness and reliability of immobilization techniques (Truett, Holberger, and Sanning, 1982).

Immobilization methods which are currently being investigated include: precipitation, chelation, and polymerization.

Precipitation is the most promising method for immobilizing dissolved metals such as lead, cadmium, zinc, and iron. Some forms of arsenic, chromium, mercury, and some organic fatty acids can also be treated by precipitation (Huibregtse and Kastman, 1979). All of the divalent metal cations can be precipitated using sulfide, phosphate, hydroxide, or carbonate. However, the solubility product and the stability of the metal complexes vary. Because of the low solubility product of sulfides and the stability of the metal sulfide over a broad pH range, sulfide precipitation looks most promising. The remaining anions decrease in effectiveness in the following order: phosphate > hydroxide > carbonate. Metal carbonates and hydroxides are stable only over a narrow pH range and the optimum pH range varies for different metals. Precipitation of the metal as the metal phosphate may require very high concentration of orthophosphate since calcium and other naturally occurring soil cations present in high concentrations will precipitate first.

Sodium sulfate used in conjunction with sodium hydroxide has shown widespread applicability for precipitatiion of metals. Precipitation takes place at a neutral or slightly alkaline pH. Resolubilization of sulfides is low. Addition of sodium hydroxide minimizes the formation of hydrogen sulfide gas by assuring an alkaline pH. Experiments with sulfide precipitation of zinc indicate that a high residual of unreacted sulfide may remain in solution.

As with other in-situ techniques, precipitation is most applicable to sites with sand or coarse silt strata. Disadvantages include the injection of a potential groundwater pollutant; the potential for formation of toxic gases (in the case of sulfide treatment); the potential for clogging soil pore space; and the possibility of precipitate resolubilization.

The use of chelating agents may also be a very effective means of immobilizing metals although considerable research is needed in this area. Depending upon the specific chelating agent, stable metal chelates may be highly mobile (as described in Section 9.2.3) or may be strongly sorbed to the soil. Tetran is an example of a chelating agent which is strongly sorbed to clay in soils (USEPA, 1984a).

A third method for immobilizing metals applies specifically to chromium and selenium. These metals can be present in the highly mobile, hexavalent state but can be reduced to less mobile Cr (III) and Se (IV) by addition of ferrous sulfate. Arsenic exists in soils as either arsenate, As (V), or as arsenite, As (III), the more toxic and soluble form. Arsenic can be effectively immobilized by oxidizing As (III) to As (IV) and treating the As (IV) with ferrous sulfate to form highly insoluble Fe-AsO_4.

Polymerization involves injection of a catalyst into a groundwater plume to cause polymerization of an organic monomer (e.g., styrene, vinyl chloride, isoprene, methyl methacrylate, and acrylonitrile). The polymerization reaction transforms the once fluid substance into a gel-like, non-mobile mass. In-situ polymerization is a technique most suited for groundwater cleanup following land spills or underground leaks of pure monomer. Applications for uncontrolled hazardous waste sites are very limited. Major disadvantages include very limited application and difficulty of initiating sufficient contact of the catalyst with the dispersed monomer (Huibregtse and Kastman, 1979). In-situ polymerization was successfully performed to remedy an acrylate monomer leak, in which 4,200 gallons of acrylate monomer leaked from a corroded underground pipeline into a glacial sand and gravel layer. Soil borings indicated that as much as 90 percent of the monomer had been polymerized by injection of a catalyst, activator, and wetting agent (Williams, 1982).

In-situ treatment of a leachate plume using precipitation or polymerization techniques probably has limited application. Problems associated with these techniques include:

- Need for numerous, closely-spaced injection wells even in coarse-grained deposits because the action of precipitation or polymerization will lower hydraulic conductivities near injection wells reducing treatment effectiveness

- Contaminants are not removed from the aquifer or some chemical reactions can be reversed allowing contaminants to again migrate with groundwater flow

- Injection of a potential groundwater pollutant or the formation of toxic byproducts.

Therefore, prior to the application of an in-situ precipitation or polymerization technique at a hazardous waste site, thorough laboratory- and pilot-scale testing should be conducted to determine deleterious effects and assure complete precipitation or polymerization of the chemical compounds.

Solidification methods used for chemical soil consolidation can also immobilize contaminants. Solidification and stabilization techniques are assessed in terms of their applicability for in-situ treatment of landfilled wastes in Guide to the Disposal of Chemically Stabilized and Solidified Wastes (USEPA, 1982). The assessment concluded that most work with these techniques has not involved in-situ treatment; most are not applicable to hazardous waste sites, and most of the techniques involve a thorough mixing of the solidifying agent and the waste (Truett, Holberger, and Sanning, 1982). Injection of silicate gel may be feasible to immobilize subsurface contaminants, but may negatively impact groundwater quality (Truett, Holberger, and Sanning, 1982).

9.2.5 Detoxification

In-situ treatment techniques discussed in this section are those which serve to destroy, degrade, or otherwise reduce the toxicity of contaminants and include neutralization, hydrolysis, oxidation/reduction, enzymatic degradation, and permeable treatment beds. These methods are applicable to specific chemical contaminants, therefore, use of these in-situ techniques at waste sites will be limited.

Neutralization involves injecting dilute acids or bases into the groundwater to adjust the pH. This pH adjustment can serve as pretreatment prior to in-situ biodegradation, oxidation, or reduction to optimize the pH range. It can be used to neutralize acidic or basic plumes that need no other treatment, or to neutralize groundwater following another treatment. It can also be used during oxidation, reduction, or precipitation to prevent the formation of toxic gases including hydrogen sulfide and hydrogen cyanide.

The pH adjustment can also be used to increase the hydrolysis rate of certain organics. Hydrolysis involves the displacement of a group on an organic moiety with a hydroxyl group from water, according to the displacement reaction:

$$RX + H_2O \rightarrow ROH + HX$$

in which R is the organic moiety and X is the leaving group. Of the parameters which affect the rate of hydrolysis (temperature, solvent composition, catalysis, and pH), pH adjustment has the greatest potential. The rate of hydrolysis can be increased up to one order of magnitude for a change of one standard unit in pH (USEPA, 1985). Classes of compounds with potential for in-situ degradation by hydrolysis include:

- Esters
- Amides
- Carbamates
- Phosphoric and phosphonic acid esters
- Pesticides.

Because a hydrolysis product may be more toxic than the present compound, the pathways for reactions must be determined to ensure toxic products are not produced. USEPA (1985) has a more thorough discussion of this technology.

Many of the environmental, health, and safety considerations that apply to solution mining also apply here. In contrast to solution mining, in-situ neutralization/detoxification techniques do not inherently incorporate collection systems. However, a collection system should be incorporated as a fail safe measure, to prevent migration of the treatment reagents and any contaminants which are not successfully treated.

Oxidation and reduction reactions serve to alter the oxidation state of a compound through loss or gain of electrons, respectively. Such reactions can detoxify, precipitate, or solubilize metals, and decompose, detoxify, or solubilize organics. Oxidation may render organics more amenable to biological degradation. As with many of these chemical treatment technologies, oxidation/reduction techniques are standard wastewater treatment approaches, but their application as in-situ treatment technologies is largely conceptual.

Oxidation of inorganics in soils, is for all practical purposes limited to oxidation of arsenic and possibly some lead compounds. The in-situ oxidation of arsenic compounds with potassium permanganate ($KMnO_4$) has been used to successfully reduce the arsenic concentrations in groundwater in the vicinity of a zinc ore smelter near Cologne, Germany (Stief, 1984). 64,000 lb. of $KMnO_4$ were injected into 17 wells and piezometer wells resulting in an average decrease in arsenic groundwater concentrations from 13.6 mg/l to 0.06 mg/l from 1975 to 1977. In 1979, however, an increase indicated that the mixing of groundwater and $KMnO_4$ had not been thorough.

Of the numerous oxidizing agents available, three have been considered potentially useful in the in-situ detoxification of organics groundwater and soils contaminated with organics: hydrogen peroxide, ozone, and hypochlorites (USEPA, 1985). Each can react with a broad range of organics and could potentially oxidize a number of different organic contaminants in a hazardous waste site. Selection of the appropriate oxidizing agent is dependent in part upon the substance or substances to be detoxified, but also upon the feasibility of delivery and environmental safety. Although there are some compounds that will not react with hydrogen peroxide but will react with ozone or hypochlorite, hydrogen peroxide appears to be the most feasible for in-situ treatment.

Ozone gas is a very strong oxdizing agent that is very unstable and extremely reactive. It cannot be shipped or stored; therefore it must be generated on-site prior to application. Ozone rapidly decomposes and its half-life in groundwater is only 18 minutes (USEPA, 1985). Ozone is used in the treatment of drinking water, municipal wastewater, and industrial waste, but has never been used in the treatment of contaminated soils or groundwater. Ozone oxidation with ultraviolet irradiation successfully reduced concentrations of benzene, phenols, and trichloroethylene in lake water (Glaze et al., 1980).

Hypochlorite, generally available as potassium, calcium, or sodium hypochlorite (bleach) is also used in the treatment of drinking water, municipal wastewater, and industrial waste. Hypochlorites have never been used in the treatment of contaminated groundwater or soils. Tolman et al. (1978) has described the conceptual design and in-situ detoxification of cyanide with sodium hypochlorite. The reaction of many organics with hypochlorite results in the formation of chlorinated organics which can be as or more toxic than the original contaminant. The formation of lower molecular weight chlorinated organics (e.g., trihalomethanes) in drinking water from hypochlorite treatment for disinfection purposes has become a major concern of the drinking water industry.

Hydrogen peroxide (H_2O_2), a moderate strength chemical oxidant, is used routinely in municipal wastewater treatment to control various factors of biological treatment, and is also used in industrial waste treatment to detoxify cyanide and various organic pollutants. Table 9-17, developed by USEPA (1985), indicates chemical compound classes that may be degraded using hydrogen peroxide.

Hydrogen peroxide is commercially available in aqueous solutions of several concentrations and is miscible in water at all concentrations. It has been delivered successfully in dilute solutions to the subsurface as an oxygen source in a bioreclamation project (Raymond, Jamison, and Hudson, 1984) (see Section 9.1). One supplier has developed a line of hydrogen peroxide solutions specifically designed for reclamation purposes (FMC, 1985).

Chemical reduction is the process by which the oxidation state of a compound is reduced. Reducing agents are election donors, with reduction accomplished by the addition of elections to the atom.

Chemical reduction does not appears as promising as oxidation for the treatment or organics. Although reseaches have demonstrated reductive dehalogenation of a variety of chlorinated organics and reduction of unsaturate aromatics and aliphatics in laboratory studies using catalyzed metal powders the treatment reagents are costly and the effectiveness of chemical reduction in soils has not been demonstrated.

Chemical reduction does, however, appear promising for treatment of chromium and selenium in soils. The in-situ reduction of hexavalent to divalent chromium has been accomplished in Arizona well water using minute quantities of reducing agent. (Srivastava and Haji-Djafari, 1983).

There are a number of disadvantages with the use of oxidizing and reducing agents which limit their use at hazardous waste sites. The treatment compounds are non-specific and this may result in degradation of non-targeted compounds. There is the potential, particularly with oxidation, for the formation of more toxic or more mobile degradation products. Also, the introduction of these chemicals into the groundwater system may create a pollution problem in itself. As with soil flushing, uncertainty exists with respect to obtaining adequate contact with the contaminants in the plume.

Enzymatic degradation of organics with cell-free enzymes holds potential as a possible in-situ treatment technique. Purified enzyme extracts, harvested from microbial cells, are commonly used in industry to catalyze a variety of reactions, including the degradation of carbohydrates and proteins. A bacterial enzyme preparation has been used to detoxify organophosphate waste from containers (Munnecke, 1980). Parathion hydrolase has been tested under field conditions in the degradation of the pesticide diazinon (Paulson et al., 1984). The studies indicate that parathion hydrolase can be used to effectively reduce rapidly large concentrations of diazinon in soil. The enzyme is readily soluble in water, is reasonably stable at summer temperatures, and can be easily handled in the field. The pH or organic content of the soil does not appear to affect the enzyme's effectiveness. It appears from the study

TABLE 9-17.
WASTE CHEMICAL CLASSES ABILITY TO REACT WITH HYDROGEN PEROXIDE

Chemical Compound	Yes	No	Unknown	Comments
Aliphatic Hydrocarbons	x	x		saturated alkanes unreactive; unsaturated compounds form epoxides and poly-hydroxy compounds
Alkyl Halides			x	
Ethers			x	
Halogenated Ethers and Epoxides			x	
Alcohols	x			require Fe^{+2} catalyst
Glycols, Epoxides	x			
Aldehydes, Ketones	x			may require catalyst
Carboxylic Acids	x			
Amides	x			
Esters			x	
Nitriles	x			
Amines	x			
Azo Compounds, Hydrazine Derivatives	x			
Nitrosamines	x			
Thiols	x			may require catalyst
Sulfides, Disulfides	x			may require catalyst
Sulfonic Acids, Sulfoxides			x	
Benzene and substituted Benzene	x			require Fe^{+2}
Halogenated Aromatic Compounds			x	
Nitrophenolic Compounds	x			
Fused Polycyclic Hydrocarbons	x			
Fused Non-Alterant Polycyclic Hydrocarbon	x			
Heterocyclic Nitrogen Compounds	x			
Heterocyclic Oxygen Compounds	x			
Heterocyclic Sulfur Compounds			x	
Organophosphorus Compounds			x	

Source: USEPA, 1985

that parathion hydrolase could be used effectively to clean up diazinon spills, but more research is required to determine how to obtain optimal field efficacy of the enzyme.

Permeable treatment beds are essentially excavated trenches placed perpendicular to groundwater flow and filled with an appropriate material to treat the plume as it flows through the material (see Figure 9-7). Some of the materials that may be used in the treatment bed are limestone, crushed shell, activated carbon, glauconitic green sands, and synthetic ion exchange resins. Permeable treatment beds have the potential to reduce the quantities of contaminants present in leachate plumes. The system is applicable to relatively shallow groundwater tables containing a plume. To date, the application of permeable treatment beds at hazardous waste sites has not been performed. However, bench- and pilot-scale testing has provided preliminary quantification of treatment bed effectiveness. Potentially numerous problems exist in using a permeable treatment bed. These include saturation of bed material, plugging of bed with precipitates, and short life of treatment materials. Therefore, permeable treatment should probably be considered as a temporary remedial action rather than a permanent one.

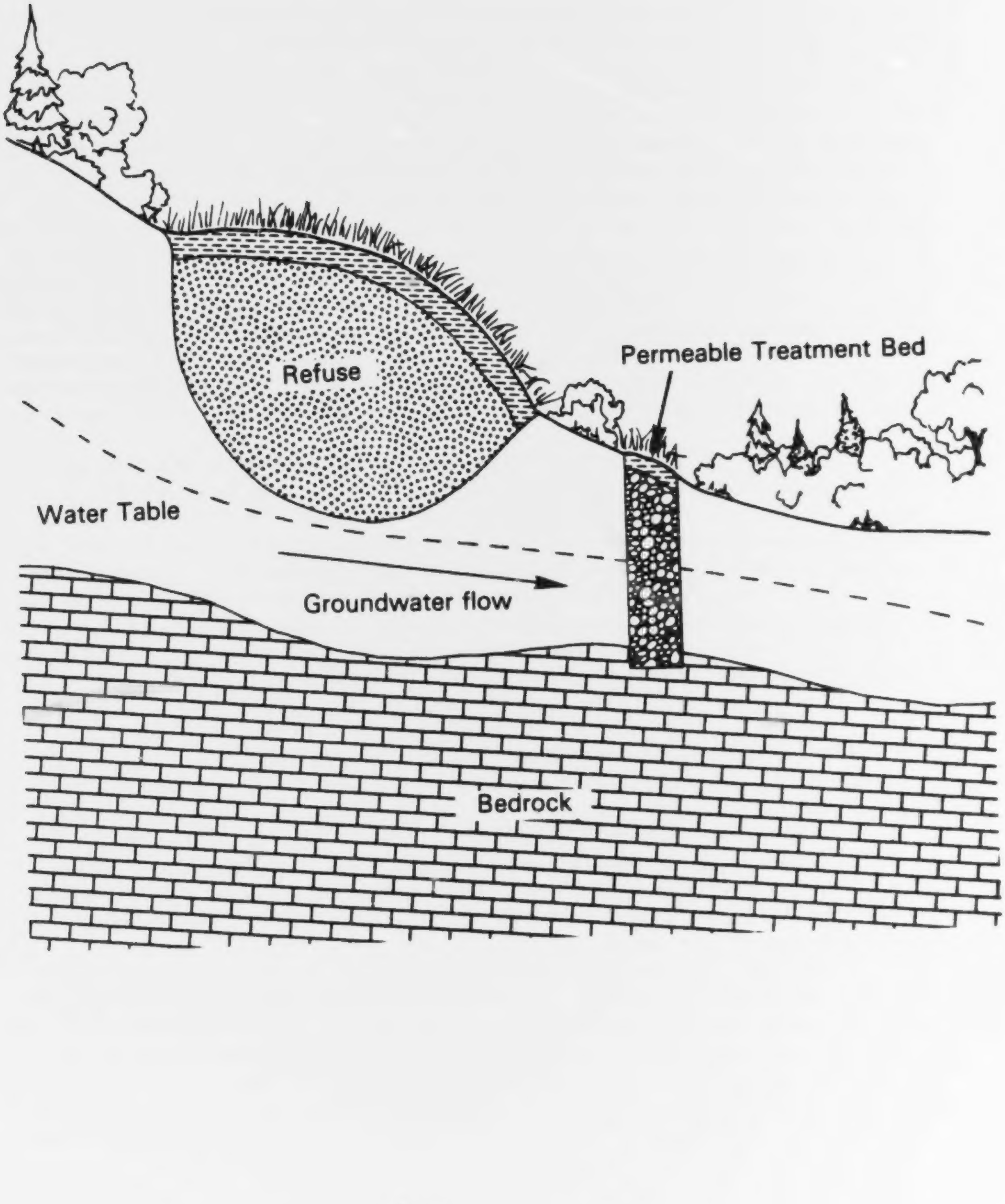
A limestone or crushed shell bed can be used to neutralize acidic groundwater and retain certain metals such as cadmium, iron, and chromium. The effectiveness of limestone as a barrier depends primarily on the pH and volume of the solution passing through the limestone (Artiola and Fuller, 1979). The nature of the heavy metal is also an important factor. A laboratory study demonstrated that limestone was more effective at retaining chromium III than for chromium VI and other metals (Artiola and Fuller, 1979).

Fuller and other researchers (USEPA, 1978) have discussed the use of crushed limestone as an effective, low cost landfill liner to aid in attenuating the migration of certain heavy metals from solid waste leachates. According to the authors, dolomitic limestone (containing significant amounts of magnesium carbonate) is less effective in removing ions than purer limestone containing little magnesium carbonate. Therefore, in the design of a limestone treatment bed, limestone with high calcium content is recommended to remove heavy metals and to neutralize contaminated groundwater.

In regard to designing vertical permeable treatment beds, the particle size of the limestone used should be selected dependent on the type of soil in which groundwater flows (i.e., which controls flow rates) and the level of contamination. In general, a mixture of gravel-size and sand-size limestone should be used to minimize settling through dissolution. Where excessive channelling through the bed by rapid groundwater movement is expected or where improved contact time between the contaminated groundwater and the treatment bed is required, a higher percentage of sand-size particles is more appropriate.

A variation on the use of limestone permeable treatment beds to neutralize plumes is the use of limestone or crushed shell layered over a waste site to indefinitely stabilize the disposed waste. This approach will be used to

FIGURE 9-7
INSTALLATION OF A PERMEABLE TREATMENT BED



reduce solubility of metal hydroxides by maintaining highly alkaline conditions in the waste (Francis, 1984).

Activated carbon as a possible treatment bed material has the capability of removing nonpolar organic compounds from contaminated plumes, but is not practical for the removal of heavy metals. Activated carbon will not remove polar organics. However, the high cost of activated carbon, the potential for desorption of adsorbed compounds, and the likelihood of a short bed-life in the presence of high waste concentrations make the use of activated carbon beds cost-prohibitive under most circumstances.

Glauconitic greensands have potential for the removal of heavy metals. Bench-scale studies with leachate indicate that the highest removal efficiencies are for copper, mercury, nickel. arsenic, and cadmium and that efficiencies increase with contact time (Spoljaric, N. Delaware Geological Survey, Newark, DE, personal communication, 1980). With contact time in the field being on the order of days, metal removal efficiencies may be extremely high. Experiments indicate that the greensands may also have a high capacity for heavy metal cation retention, even when flushed with solutions of highly alkaline or acidic pH (Spoljaric, N. Delaware Geological Survey, Newark, DE, personal communication, 1980). An in-situ experiment in England (Ross, 1980) demonstrated promising retention capabilities. Glauconitic greensands appear promising; however, more research is required to determine their sorptive capacity and capability for treating higher concentrations of heavy metals.

Advantages of glauconitic treatment beds, based on studies to date, include good permeability, abundance in the Atlantic Coastal Plain (i.e., New Jersey, Delaware, and Maryland), effectiveness in removal and retention of many heavy metals, and good retention time characteristics for efficient treatment. Among the disadvantages of using glauconitic treatment beds are unknown saturation characteristics and potential for plugging over time, potential reduction in pH, limited to areas of natural occurrence such as the mid-Atlantic region, and possibility of land purchase requirements as glauconite is not commercially mined.

9.2.6 Technology Selection/Evaluation

This section described a wide range of chemical and in-situ treatment methods, therefore, generalizations regarding the feasibility and effectiveness of these methods are not possible. However, all of these methods are developmental or conceptual and none have been fully demonstrated for hazardous waste site remediation.

Of all the methods described, soil flushing methods involving the use of water surfactants appear to be most feasible and cost-effective for organics. They can use relatively cheap, innocuous treatment reagents, can be used to treat a broad range of waste constituents, and do not result in toxic degradation products.

The most feasible methods for treating inorganics in-situ include soil flushing with dilute acids, chelating agents or other treatment agents which will mibolize the metals. Precipitation of the metals as sulfides or phosphates, and the use of permeable treatment beds have potential application although there are potentially serious drawbacks with each of these methods. For example, permeable treatment beds are prone to clogging, and sulfide precipitation can clog soils and reduce permeability.

Mention of these potential drawbacks should not preclude consideration of in-situ methods. However, laboratory- and possibly pilot-scale testing is likely to be required in each case, resulting in delays in implementing the remedial action.

As with biological treatment methods, chemical treatment methods used to deliver and recover treatment reagents also affect the reliability of these methods. Reliability of pumping and subsurface drainage systems have been described previously.

Again, worker safety considerations are the same as those related to in-situ biological treatment methods involving a potentially hazardous chemical reagent. The same precautions required when working with hydrogen peroxide (Section 9.1.6) are required when working with acids, bases, surfactants, and other potentially hazardous reagents.

9.2.7 Costs

Costs for the chemical in-situ treatment approaches discussed in this section are difficult to estimate since these methods have largely not been demonstrated at hazardous waste sites and no actual cost data are available. In-situ treatment costs are variable, but could be less than excavation and removal methods and/or pump and treat methods. As with removal, in-situ approaches are conducted on a one-time basis, so there are generally no long-term operation and maintenance costs.

Costs for the chemical treatment approaches involving the delivery of a reagent to the subsurface (soil flushing, various immobilization techniques, neutralization, hydrolysis, oxidation, reduction, and enzymatic degradation) will depend on the amount of material to be treated, the amount of chemical reagent required, the costs for the delivery system (injection wells or infiltration galleries), the chemical and feed system, and fees for probing, excavation, and drilling. Costs for laboratory- and pilot-scale studies should also be considered when performing such a treatment approach. Soil flushing, which involves bringing contaminated water to the surface for treatment, would require a wastewater treatment system. Costs for the drains and pumping are presented in Sections 5.1 and 5.2, respectively. Table 9-10 provides unit costs for chemicals.

1985 unit costs for the installation of a permeable treatment bed are shown in Table 9-18. Total closure costs for stabilizing approximately 8,000

TABLE 9-18.
UNIT COSTS FOR INSTALLATION OF A PERMEABLE TREATMENT BED

Item	Assumptions	Costs
Trench excavation	20 ft deep, 4 ft wide, by backhoe	\$1.40 cubic yard[1]
Spreading	Spread by dozer to grade trench and cover	\$1/cubic yard[1]
Well-point dewatering	500 ft header 8" diameter, for one month	\$115/linear foot[1]
Sheet piling	20 feet deep; pull and salvage	\$7.70/square foot[1]
Walers, connections, struts	2/3 salvage	\$165/ton[2]
Liner	30 mil PVC 30 mil CPE	\$0.25-0.35/square foot[4] \$0.35-0.45/square foot
Limestone	Mixed "gravel size" and "sand size"	\$30-45/ton[3]
	Installation (Backfill trench, 100 foot haul)	\$3.70/cubic yard[1]

---

[1] Godfrey, 1984; Costs are total, including contractor overhead and profit.
[2] Godfrey, 1984; Materials only.
[3] Schnell, 1985.
[4] Cope, Karpinski, and Steiner, 1984.

cubic yards of sludge contaminated with nickel hydroxide by covering the site with a 1 inch layer of calcium carbonate are estimated at $100,000 to $200,000, compared with $900,000 to $1 million estimated for excavation and removal (Francis, 1984).

9.3 Physical In-Situ Methods

9.3.1 General Description

A number of methods are currently being developed which involve physical manipulation of the subsurface in order to immobilize or detoxify waste constituents. These technologies, which include in-situ heating, vitrification and ground-freezing, are in the early stages of development and detailed information is not available.

In-situ heating has been proposed as a method to destroy or remove organic contaminants in the subsurface through thermal decomposition, vaporization, and distillation. Methods recommended for in-situ heating are steam injection (Hoogendorn, 1984) and radio frequency heating (Dev, Bridges, and Sresty, 1984).

The radio frequency heating process has been under development since the 1970s. Field experiments have been conducted for the recovery of hydrocarbons. The method involves laying a row of horizontal conductors on the surface of a landfill and exciting them with an RF generator through a matching network. The decontamination is accomplished in a temperature range of 300° to 400°C, assisted with steam, and requires a residence time of about two weeks. A gas or vapor recovery system is required on the surface. Excavation, mining, drilling, or boring is not required. Field tests found that leakage radiation levels did not exceed the recommended ANSI Standard C-95. Preliminary design and cost estimates for a mobile RF in-situ decontamination process (see Section 9.2.5) indicate that the method is 2 to 4 times cheaper than excavation and incineration (Dev, Bridges, and Sresty, 1984). This method appears very promising for certain situations involving contamination with organics, although more research is necessary to verify the effectiveness in-situ.

Artificial ground freezing involves the installation of freezing loops in the ground and a self-contained refrigeration system that pumps coolant around the freezing loop (Sullivan, Lynch and Iskandar, 1984). Although never used in an actual waste containment operation, the technology is being used increasingly as a construction method in civil engineering projects. Artificial ground freezing is done not on the waste itself, which may have a freezing point much lower than that of the soil systems, but on the soil surrounding the hazardous waste. It renders the soil practically impermeable, but is useful only as a temporary treatment approach because of the thermal maintenance expense (Sullivan, Lynch, and Iskandar, 1984).

In-situ vitrification is a technology being developed for the stabilization of transuranic contaminated wastes, and is conceivably applicable to other hazardous wastes (Fitzpatrick et al., 1984). Several laboratory-scale and pilot-scale tests have been conducted, and a large-scale testing system is currently being fabricated. The technology is based upon electric melter technology, and the principle of operation is joule heating, which occurs when an electrical current is passed through a molten mass. Contaminated soil is converted into durable glass, and wastes are pyrolyzed or crystallized. Off-gases released during the melting process are trapped in an off-gas hood. The depth of the waste is a significant limiting factor in the application of this technology: 1 to 1.5 meters of uncontaminated overburden lowers release fractions considerably (Fitzpatrick et al., 1984).

Cost estimates for five in-situ vitrification large-scale configurations are provided in Table 9-19 (Fitzpatrick et al., 1984, citing Oma et al.). These cost comparisons are based on vitrifying to a depth of 15 feet. Of these costs, approximately 30 to 46 percent is for power, 10 to 13 percent is for equipment, 36 to 45 percent is for labor, and 5 to 10 percent is for electrodes (Sanning, 1984). Soil moisture can increase operating costs by increasing requirements because the water in the soil must be evaporated. Estimates for treating a humid site in the eastern United States is near $85/cubic foot (1982 dollars) (Sanning, 1984).

The cost of a hypothetical hazardous waste site decontaminated by radio frequency heating was estimated for a 1 acre landfill area with contamination extending to 20 feet (Dev, Bridges, and Sresty, 1984). Volatile matter in the landfill was assumed to range from 5 to 20 percent by weight, with 10 percent of the total volatile matter being organic. Total capital costs for a purchase power option was estimated at approximately $17 million. The capital costs increase to $27.5 million if the power is generated on-site. Capital costs for equipment for excavation and incineration were estimated at $832,000. The total costs of decontamination (including operating costs but not capital costs) was estimated to be between $4.6 and $5.7 million for radio frequency heating (purchased power plant). Total treatment costs for incineration (excluding capital costs) were estimated at between $9 and $25.2 million (Dev, Bridges, and Sresty, 1984).

Figure 9-8 provides estimated costs for ground freezing plotted as a function of freezing rod space for a hypothetical site. The site requires a 1,000 foot frozen wall which is 3 feet thick and is placed down to depth of bedrock (40 feet). The site is assumed to be located in coarse quartz sand, 150 miles from the drilling and refrigeration contractors. From Figure 9-5, one can see that as the drill space becomes tighter, the fuel costs, equipment rentals, and time for wall completion are reduced. A tight drill space yields small frozen soil column radii and permits use of less expensive refrigeration equipment. The drawback of close drill spacing is the expense associated with the drilling operation. The linear footage of piping, a drive shoe for each well drilled, and the labor charge per vertical foot drilled overwhelm all other economic parameters. Analysis of costs for this hypothetical site illustrated that ground/freezing is only applicable as a short-term remedial measure (Sullivan, Lynch, and Iskandar, 1984).

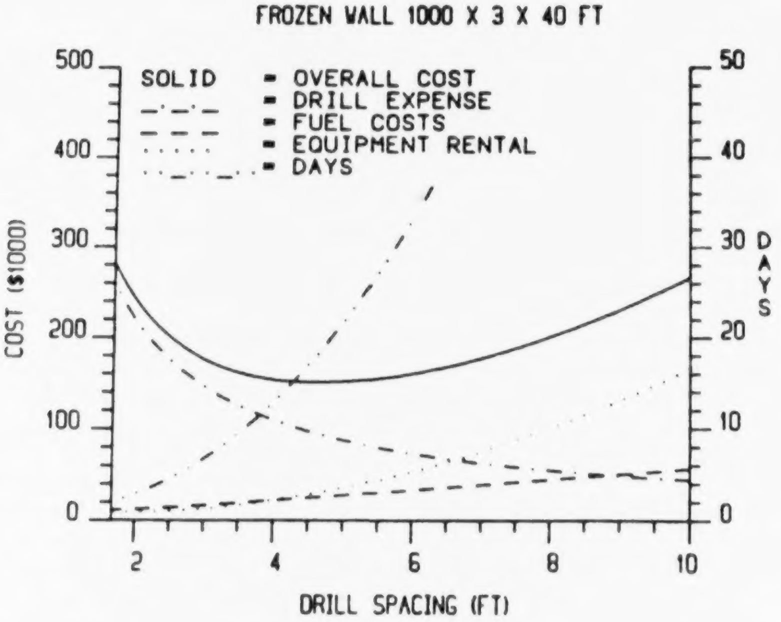
TABLE 9-19.

1982 COST ESTIMATES FOR FIVE IN-SITU VITRIFICATION LARGE-SCALE CONFIGURATIONS

Number	Site	Power	Heat Loss	Manpower Level	Total Cost of Soil Vitrified	Total Cost of Soil Vitrified
1	Hanford	Local	High	Average	$\$187/m^3$	$\$5.30/ft^2$
2	Hanford	Local	Average	Average	$\$161/m^3$	$\$4.60/ft^2$
3	Hanford	Local	Average	Above Average	$\$183/m^3$	$\$5.20/ft^2$
4	Generic	Local	Average	Average	$\$180/m^3$	$\$5.10/ft^2$
5	Generic	Portable	Average	Average	$\$224/m^3$	$\$6.30/ft^2$

Source: Fitzpatrick et al., 1984

FIGURE 9-8
ECONOMIC OVERVIEW OF GROUND FREEZING FOR A HYPOTHETICAL SITE



Source: Sullivan, Lynch, and Iskandar, 1984

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SECTION 10

DIRECT WASTE TREATMENT

This section describes direct waste treatment methods applicable for treating aqueous, gaseous, and solid waste streams produced at hazardous waste sites. Many of the treatment methods described in this section are widely used in industrial waste treatment applications and information on their design and construction is well described in the literature cited throughout the section. As a result detailed information pertaining to design and construction has not been included. Instead, this section emphasizes applications and limitations of these methods for hazardous waste treatment. Section 10.1 describes aqueous waste treatment methods. Section 10.2 describes methods for solid waste treatment including solids separation and dewatering methods. Section 10.3 addresses solidification and stabilization technologies applicable for liquid and solid wastes. Commonly used methods for treating gaseous emissions (with the exception of incineration) are addressed in Section 10.4 Incineration and other thermal treatment methods are addressed separately in Section 10.5 since these methods apply to liquid, solid, and gaseous wastes.

10.1 Aqueous Waste Treatment

Aqueous waste streams resulting from the clean up of hazardous waste sites vary widely with respect to volume, level, and type of contaminants and level of solids. The major sources of aqueous wastes include:

- Leachate plumes which have been pumped to the surface or collected via subsurface drains
- Contaminated water generated during dredging operations
- Contaminated run-off collected in impoundments or basins
- Contaminated water generated from equipment cleanup
- Aqueous waste generated from sediment or sludge dewatering
- Highly concentrated wastewater streams generated from certain aqueous waste treatment processes (e.g., backwash from filtration, concentrate from reverse osmosis).

Because these waste streams are so diverse in volume, type, and concentration of contaminants, a wide variety of treatment processes will have application to hazardous waste site cleanup. This section addresses those processes which are considered most applicable for hazardous waste site remediation. Rarely will any one unit treatment process be sufficient for aqueous waste treatment. Therefore, the discussions which follow include information on unit treatment processes which are frequently used in combination and any pretreatment requirements which are a prerequisite to effective use of each treatment process. The unit treatment processes considered in this section include:

- Activated carbon
- Activated sludge
- Filtration
- Precipitation/flocculation
- Sedimentation
- Ion exchange
- Reverse osmosis
- Neutralization
- Gravity separation
- Air stripping
- Chemical oxidation
- Chemical reduction.

Aqueous waste treatment at hazardous waste sites can be accomplished using one of four general approaches:

- On-site treatment using mobile treatment system
- On-site construction and operation of treatment systems
- Pretreatment followed by discharge to a POTW
- Hauling of waste to an off-site treatment facility.

Mobile treatment systems and systems constructed on-site have broadest applicability. Wastewaters discharged to POTWs often require extensive pretreatment in order for the facility to meet its NPDES permit conditions. Other factors which determine the feasibility of POTW discharge include whether the facility has the hydraulic capacity to handle the waste, whether accepting the waste will result in additional monitoring requirements or process changes, and the potential for opposition in the community.

Hauling wastes off-site for treatment is limited to all but very small wastewater volumes.

10.1.1 Activated Carbon Treatment

10.1.1.1 General Description

The process of adsorption onto activated carbon involves contacting a waste stream with the carbon, usually by flow through a series of packed bed reactors. The activated carbon selectively adsorbs hazardous constituents by a surface attraction phenomenon in which organic molecules are attracted to the internal pores of the carbon granules.

Adsorption depends on the strength of the molecular attraction between adsorbent and adsorbate, molecular weight, type and characteristic of adsorbent, electrokinetic charge, pH, and surface area.

Once the micropore surfaces are saturated with organics, the carbon is "spent" and must either be replaced with virgin carbon or removed, thermally regenerated, and replaced. The time to reach "breakthrough" or exhaustion is the single most critical operating parameter. Carbon longevity balanced against influent concentration governs operating economics.

Most hazardous waste treatment applications involve the use of adsorption units which contain granular activated carbon (GAC) and operate in a downflow series mode such as that shown in Figure 10-1 (Brunotts et al., 1983).

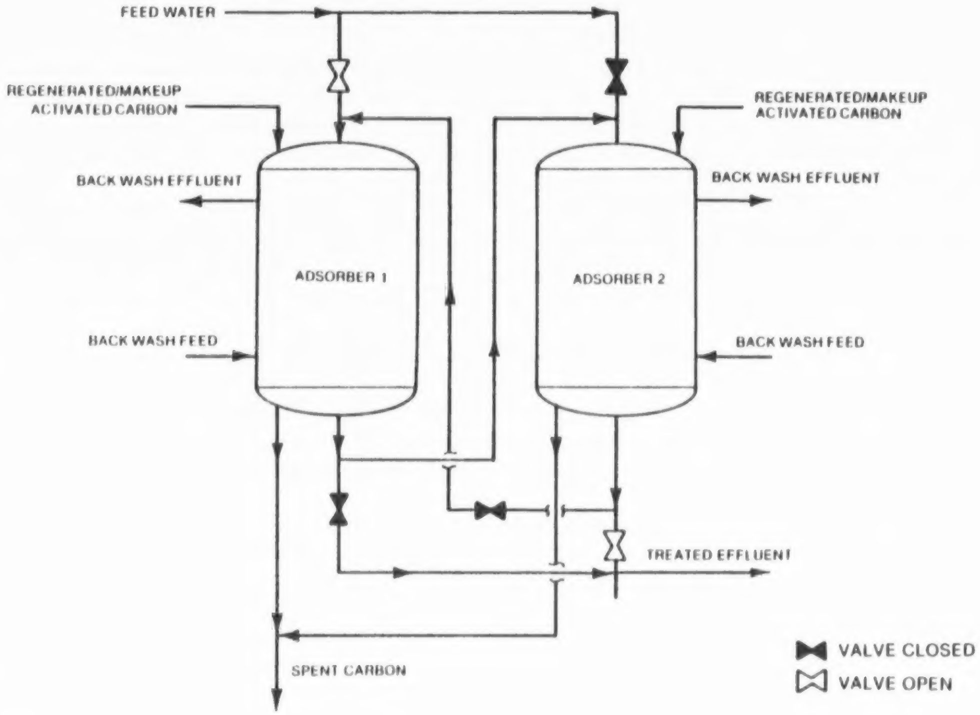
The downflow fixed bed series mode has been found to be generally most cost-effective and produces the lowest effluent concentrations relative to other carbon adsorber configurations (e.g., downflow in parallel, moving bed, upflow-expanded). The units may be connected in parallel to provide increased hydraulic capacity.

10.1.1.2 Applications/Limitations

Activated carbon is a well developed technology which is widely used in the treatment of hazardous waste streams. It is especially well suited for removal of mixed organics from aqueous wastes. Table 10-1 provides an indication of the treatability of organics commonly found in groundwater.

Table 10-2 delineates various factors which influence the applicability of activated carbon treatment for any given waste (Nalco Chemical Co., 1979). As carbon adsorption is essentially an electrical interaction phenomenon, the polarity of the waste compounds will largely determine the effectiveness of the adsorption process. Highly polar molecules cannot be effectively removed by carbon adsorption. Another factor to consider in determining the likely effectiveness of carbon adsorption is aqueous solubility. The more hydrophobic (insoluble) a molecule is, the more readily the compound is adsorbed. Low solubility humic and fulvic acids which may be present in the groundwater can sorb to the activated carbon more readily than most waste contaminants and result in rapid carbon exhaustion.

FIGURE 10-1.
TWO-VESSEL GRANULAR CARBON ADSORPTION SYSTEM



Source: USEPA, 1973a

In addition, some metals and inorganic species have shown excellent to good adsorption potential, including antimony, arsenic, bismuth, chromium, tin, silver, mercury, cobalt, zirconium, chlorine, bromine, and iodine.

Carbon adsorption is frequently used following biological treatment and/or granular media filtration in order to reduce the organic and suspended solids load on the carbon columns, or to remove refractory organics which cannot be biodegraded. Air stripping may also be applied prior to carbon adsorption in order to remove a portion of the volatile contaminants, thereby reducing the organic load to the column. These pretreatment steps all minimize carbon regeneration costs.

The highest concentration of solute in the influent stream that has been treated on a continuous basis is 10,000 ppm total organic carbon (TOC), and a 1 percent solution is currently considered as the upper limit (De Renzo,

TABLE 10-1.

FACTORS AFFECTING EQUILIBRIUM ADSORBABILITY

- Compound Adsorbability Favored by:
  - Increasing carbon chain length[a]
  - Increasing aromaticity
  - Decreasing polarity
  - Decreasing branching
  - Decreasing solubility
  - Decreasing degree of dissociation
- Functionality
  - Relative adsorbability: acids > aldehydes > esters > ketones > alcohols > glycols when number of carbon atoms is <4
- pH Effects
  - Undissociated species are more easily adsorbed
    - low pH favors adsorption of acids (e.g., volatile acids, phenol)[b]
    - high pH favors adsorption of bases (e.g., amines)
  - Other compounds: adsorption can be favored by higher pH
    - Postulated general effect:
      - Partial neutralization of surface acidity reduces hydrogen-bonding of surface groups eliminating steric blockage of micropores
- Temperature
  - Increased temperatures can increase rate of adsorption due to viscosity and diffusivity effects
  - Exothermic adsorption reactions are favored by decreasing temperatures, usually a minor effect on equilibrium level

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[a] When the rate is controlled by intraparticle transport, decreasing molecular size would result in faster rate, all else being equal.

[b] This often is the most significant pH effect, so adsorption generally is increased with decreasing pH.

Source: Conway and Ross, 1980.

TABLE 10-2.

CARBON INFLUENT AND EFFLUENT

Organic Compounds in Groundwater	Number of Occurrences	Influent* Concentration Range	Carbon Effluent* Concentration Achieved
Carbon tetrachloride	4	130 ug/l-10 mg/l	<1 ug/l
Chloroform	5	20 ug/l-3.4 mg/l	<1 ug/l
DDD	1	1 ug/l	<.05 g/l
DDE	1	1 ug/l	<0.05 ug/l
DDT	1	4 ug/ll	<0.05 ug/l
CIS-1,2-dichloroethylene	8	5 ug/l-4 mg/l	<1 ug/l
Dichloropentadiene	1	450 ug/l	<10 ug/l
Disopropyl ether	2	20-34 ug/l	<1 ug/l
Tertiary methyl-butylether	1	33 ug/l	<5.0 ug/l
Diisopropyl methyl phosphonate	1	1,250 ug/l	<50 ug/l
1,3-dichloropropene	1	10 ug/l	<1 ug/l
Dichlorethyl ether	1	1.1 mg/l	<1 ug/l
Dichloroisopropylether	1	0.8 mg/l	<1 ug/l
Benzene	2	0.4-11 mg/l	<1 ug/l
Acetone	1	10-100 ug/l	<10 mg/l
Ethyl acrylate	1	200 mg/l	<1 mg/l
Trichlorotrifloroethane	1	6 mg/l	<10 ug/l
Methylene chloride	2	1-21 mg/l	<100 ug/l
Phenol	2	63 mg/l	<1 ug/l
Orthochlorophenol	1	100 mg/l	<1 mg/l
Tetrachloroethylene	10	5 ug/l-70 mg/l	<1 ug/l
Trichloroethylene	15	5 ug-16 mg/l	<1 ug/l
1,1,1-trichloroethane	6	60 ug/l-25 mg/l	<1 ug/l
Vinylidiene chloride	2	5 ug/l-4 mg/l	<1 ug/l
Toluene	1	5-7 mg/l	<10 ug/l
Xylene	3	0.2-10 mg/l	<101 ug/l

*Analyses conducted by Calgon Carbon Corporation conformed to published U.S. EPA protocol methods. Tests in the field were conducted using available analytical methods.

Source: O'Brien and Fisher, 1983

1978). Pretreatment is required for oil and grease and suspended solids. Concentrations of oil and grease in the influent should be limited to 10 ppm. Suspended solids should be less than 50 ppm for upflow systems, while downflow systems can handle much higher solids loadings.

10.1.1.3 Design Considerations

The phenomenon of adsorption is extremely complex and not mathematically predictable. To accurately predict performance, longevity and operating economics, field pilot plant studies are necessary.

In order to conduct an initial estimate of carbon column sizing, the following data need to be established during pilot plant testing:

- Hydraulic retention time (hours)
- Flow (gallons/minute)
- Hyraulic capacity of the carbon (gallons waste/pound carbon)
- Collected volume of treated waste at breakthrough (gallons)
- Carbon density (pounds carbon/cubic foot).

In the above data list, the term "breakthrough" refers to the moment when the concentration of solute being treated first starts to rise in the carbon unit effluent. The term "exhaustion" refers to the moment when the concentration of solute being treated is the same in both effluent and influent.

10.1.1.4 Technology Selection/Evaluation

Activated carbon is an effective and reliable means of removing low solubility organics. It is suitable for treating a wide range of organics over a broad concentration range. It is not particularly sensitive to changes in concentrations or flow rate and, unlike biological treatment, is not adversely affected by toxics. However, it is quite sensitive to suspended solids and oil and grease concentrations.

Activated carbon is easily implemented into more complex treatment systems. The process is well suited to mobile treatment systems as well as to on-site construction. Space requirements are small, start-up and shut-down are rapid, and there are numerous contractors who are experienced in operating mobile units.

The EPA's Mobile Physical/Chemical Treatment System includes three carbon columns that can be operated either in series or in parallel and are designed for a hydraulic loading of 200 gpm with a 27 minute contact time. This contact time has been found to be adequate for many hazardous waste streams.

However, longer contact times can be provided by reducing the hydraulic flow rate (Ghassemi, Yu, and Quinlivan, 1981).

Use of several carbon adsorption columns at a site can provide considerable flexibility. Various columns can be arranged in series to increase service life between regeneration of the lead bed or in parallel for maximum hydraulic capacity. The piping arrangement would allow for one or more beds to be regenerated while the other columns remain in service.

The most obvious maintenance consideration associated with activated carbon treatment is the regeneration of spent carbon for reuse. Regeneration must be performed for each column at the conclusion of its bed-life so the spent carbon may be restored as close as possible to its original condition for reuse; otherwise, the carbon must be disposed of. Other operation and maintenance requirements of activated carbon technology are minimal if appropriate automatic controls have been installed.

It is recommended that the thermal destruction properties of waste chemicals be determined prior to selection of activated carbon treatment technology, since any chemicals sorbed to activated carbon must eventually be destroyed in a carbon regeneration furnace. Therefore, of crucial importance to the selection of activated carbon treatment is whether the sorbed waste material can be effectively destroyed in the regeneration furnace; otherwise, upon introduction to the furnace, they will become air pollutants.

The biggest limitation of the activated carbon process is the high capital and operating cost. As described previously, the operating costs can be substantially reduced by pretreatment of the waste using biological treatment or air stripping.

10.1.1.5 Costs

The cost of activated carbon units depends on the size of the contact unit which is influenced by the concentrations of the target and non-target organic compounds in the waste stream and the desired level of target compounds in the effluent. Table 10-3 presents construction, operation and maintenance costs for cylindrical pressurized, downflow steel contactors based on a nominal detention time of 17.5 minutes and a carbon loading rate of 5gpm/ft^2. The construction costs include housing, concrete foundation, and all the necessary pipes, valves, and nozzles for operating the unit plus the initial change of carbon. The operation and maintenance cost include the electricity and assume carbon replacement once a year. However, systems for unloading spent carbon and loading fresh carbon are not included.

There are a number of manufacturers such as Calgon Carbon Corporation who market mobile activated carbon treatement systems. For example, Calgon Carbon Corporation has a trailer-mounted carbon adsorption treatment unit that can be shipped to a treatment location within 24 to 48 hours. The system can be configured wih either single or multiple pre-piped adsorber vessels. It can handle flow of up to 200 gpm. The following describes costs associated with a

Table 10-3

GENERAL COST DATA FOR VARIOUS SIZES OF ACTIVATED CARBON CONTACT UNITS

Capacity (gpm)	Column Diameter (ft)	Column Length (ft)	Housing Area (ft^2)	Construction Costs*	O&M Costs ($/yr)*
1.7	0.67	5	60	12,320	1,690
17	2	5	150	23,776	2,315
70	4	5	300	42,425	4,800
175	6.5	5	375	64,000	8,110
350	9	5	450	93,822	12,540

*Updated from 1979 to 1984 dollars using third quarter Marshall and Swift Equipment Index.

Source: Adapted from Hansen, Gumerman, and Culp, 1979.

mobile system consisting of two 10-foot diameter, 10-foot high, skid mounted vessels capable of handling up to 200 gpm (Calgon Corp, undated):

Delivery, supervision of installation and startup, tests to conduct reactivation of carbon, dismantling and removal of system (including freight to and from the site)	$25,000
Delivery and removal of one truckload of carbon (2,000 lbs)	$15,200
(Two truckloads required for a two-vessel system) - Rental fee (per month)	$5,000/month

Calgon Carbon Corporation will take spent carbon back for reactivation. Otherwise, disposal costs for spent carbon must be added.

10.1.2 Biological Treatment

10.1.2.1 General Description

The function of biological treatment is to remove organic matter from the waste stream through microbial degradation. The most prevalent form of biological treatment is aerobic, i.e., in the presence of oxygen. A number of biological treatment processes exist which may be applicable to treatment of aqueous wastes from hazardous waste sites, including conventional activated sludge, various modifications of the activated sludge process including pure oxygen activated sludge, extended aeration, and contact stabilization, and fixed film systems which include rotating biologial discs, and trickling filters.

In the conventional activated sludge process, aqueous waste flows into an aeration basin where it is aerated for several hours. During this time, a suspended active microbial population (maintained by recycling sludge) aerobically degrades organic matter in the stream along with producing new cells. A simplified equation for this process is shown below:

$$\text{Organics} + O_2 \longrightarrow CO_2 + H_2O + \text{new cells}$$

The new cells produced during aeration form a sludge which is settled out in a clarifier. A portion of the settled sludge is recycled to the aeration basin to maintain the microbial population while the remaining sludge is wasted, i.e., it undergoes volume reduction and disposal. Clarified water flows to disposal or further processing.

In the pure oxygen activated sludge process, oxygen or oxygen-enriched air is used instead of air to increase the transfer of oxygen. Extended aeration involves longer detention times than conventional activated sludge and relies on a higher population of microorganisms to degrade wastes. Contact stabilization involves only short contact of the aqueous wastes and suspended microbial solids, with subsequent settling of sludge and treatment of the sludge to remove sorbed organics. Fixed film systems involve contact of the aqueous waste stream with microorganisms attached to some inert medium such as rock or specially designed plastic material. The original trickling filter consisted of a bed of rocks over which the contaminated water was sprayed. The microbes forming a slime layer on the rocks, would metabolize the organics, while oxygen was provided as air moved countercurrent from the water flow (Canter and Knox, 1985).

Biological towers are a modification of the trickling filter. The medium (e.g., of polyvinyl chloride (PVC), polyethylene, polystyrene, or redwood) is stacked into towers which typically reach 16 to 20 ft. The contaminated water is sprayed across the top and, as it moves downward, air is pulled upward through the tower. A slime layer of microorganisms forms on the media and removes the organic contaminants as the water flows over the slime layer.

A rotating biological contactor (RBC) consists of a series of rotating discs, connected by a shaft, set in a basin or trough. The contaminated water passes through the basin where the microorganisms, attached to the discs, metabolize the organics present in the water. Approximately 40% of the disc's surface area is submerged. This allows the slime layer to alternately come in contact with the contaminated water and the air where oxygen is provided to the microorganisms (Canter and Knox, 1985).

10.1.2.2 Applications/Limitations

There is considerable flexibility in biological treatment because of the variety of available processes and adaptability of the microorganisms themselves. Many organic chemicals are considered biodegradable, although the relative ease of biodegradation varies widely. Several generalizations can be made with regard to the ease of treatability of organics by aerobic biological treatment:

- Unsubstituted nonaromatics or cyclic hydrocarbons are preferred over unsubstituted aromatics

- Materials with unsaturated bonds such as alkenes are preferred over materials with saturated bonds

- Soluble organics are usually more readily degraded than insoluble materials. Biological treatment is more efficient in removing dissolved or colloidal materials, which are more readily attacked by enzymes. This is not the case, however, for fixed film treatment systems which preferentially treat suspended matter

- The presence of functional groups affects biodegradability. Alcohols, aldehydes, acids, esters, amides, and amino acids are more degradable than corresponding alkanes, olefins, ketones, dicarboxylic acids, nitriles, and chloroalkanes

- Halogen-substituted compounds are the most refractory to biodegradation; chlorinated alphatics are generally more refractory than the corresponding aromatics, although the number of halogens and their position is also significant in determining degradation.

- Nitro-substituted compounds are also difficult to degrade although they are generally less refractory than the halogen-substituted compounds.

Although there are a number of compounds which are considered to be relatively resistant to biological treatment, it is recommended in practice that the treatability of waste be determined through laboratory Biological Oxygen Demand (BOD) tests on a case-by-case basis. Section 9.1 provides further discussion of the degradability of organics.

Despite the fact that industrial type wastes may be refractory to biological treatment, microorganisms can be acclimated to degrade many compounds that are initially refractory. Similarly, while heavy metals are inhibitory to biological treatment, the biomass can also be acclimated, within limits, to tolerate elevated concentrations of metals.

In terms of the variety of biological treatment processes available, Table 10-4 presents the applications and limitations of each. The completely mixed activated sludge process is the most widely used for treatment of aqueous wastes with relatively high organic loads. However, the high purity oxygen system has advantages for hazardous waste site remediation.

In addition, a number of other parameters may influence the performance of the biological treatment system, such as concentration of suspended solids, oil and grease, organic load variations, and temperature. Table 10-5 lists parameters that may limit system performance, limiting concentrations, and the type of pretreatment steps required prior to biological treatment.

10.1.2.3 Design Considerations

Design of the activated sludge or fixed-film systems for a particular application can be achieved best by first representing the system as a mathematical model, and then determining the necessary coefficients by running laboratory or pilot tests.

TABLE 10-4.

SUMMARY OF APPLICATIONS/LIMITATIONS FOR BIOLOGICAL TREATMENT PROCESS

PROCESS	APPLICATIONS/LIMITATIONS
Conventional	Applicable to low strength wastes; subject to shock loads
Completely-mixed Conventional	Resistant to shock loads
Extended Aeration	Requires low organic load and long detention times; low volume of sludge; available as package plant
Contact Stabilization	Not suitable for soluble BOD
Pure Oxygen	Suitable for high strength wastes; low sludge volume; reduced aeration tank volume
Trickling Filters	More effective for removal of colloidal and suspended BOD; used primarily as a roughing filter
Rotating Biological Disc	Can handle large flow variations and high organic shock loads; modular construction provides flexibility to meet increases or decreased treatment needs.

TABLE 10-5

CONCENTRATION OF POLLUTANTS THAT MAKE PREBIOLOGICAL OR PRIMARY TREATMENTS DESIRABLE

Pollutant or System Condition	Limiting Concentration	Kind of Pretreatment
Suspended solids	>50-125 mg/l flotation, lagooning	Sedimentation,
Oil or grease	>35-50 mg/l	Skimming tank or separator
Toxic ions		precipitation or ion exchange
Pb	≤ 0.1 mg/l	
$Cu+Ni+CN$	≤ 1 mg/l	
$Cr^{+6}+Zn$	≤ 3 mg/l	
Cr^{+3}	≤ 10 mg/l	
pH	<6, >9	Neutralization
Alkalinity	0.5 lb alkalinity as $CaCO_3$/lb BOD removed	Neutralization for excessive alkalinity
Acidity	Free mineral acidity	Neutralization
Organic load variation	>2:1-4:1	Equalization
Sulfides	>100 mg/l	Precipitation or stripping with recovery
Phenols	>70-300 mg/l	Extraction, adsorption, internal dilution
Ammonia	>1.6 g/l	Dilution, ion exchange, pH adjustment and stripping
Dissolved salts	>10-16 g/l	Dilution, ion exchange
Temperature	13-38°C in reactor	Cooling, steam addition

Source: Conway and Ross, 1980

The following models have been found to be reliable for designing biological treatment systems for waste streams containing priority pollutants (Cantor and Knox, 1985).

Activated Sludge:

$$V = \frac{FS_i/X}{\frac{U_{max}S_i}{S_i - S_e} - K_B}$$

Biological Tower and Rotating Biological Contactor:

$$A = \frac{FS_i}{\frac{U_{max}S_i}{S_i - S_e} - K_B}$$

where V = volume of aeration tank (ft^3)
F = flow rate (ft^3/day)
X = mixed liquor volatile solids (mg/l)
S_i = influent BOD, COD, TOC, or specific organics (mg/l)
S_e = effluent BOD, COD, TOC, or specific organics (mg/l)
U_{max} and K_B = biokinetic constants (day^{-1})
A = surface area of biological tower or rotating biological contactor (ft^2)

The biokinetic constants are determined by conducting laboratory or pilot plant studies. After the biokinetic constants are determined, the required volume of aeration tank or the required surface area for a biological tower or rotating biological contactor can be determined for any flow rate, influent concentration of BOD, COD, TOC, or specific organic, and a required effluent concentration of BOD, COD, TOC, or specific organic.

10.1.2.4 Technology Selection/Evaluation

Biological treatment has not been as widely used in hazardous waste site remediation as activated carbon, filtration and precipitation/flocculation. However, the process is well established for treating a wide variety of organic contaminants. Kincannon and Stove as reported by Canter and Knox (1985) have demonstrated the effectiveness of activated sludge for treating priority pollutants. The results shown in Table 10-6 indicate that activated sludge was effective for all groups of contaminants tested except for halogenated hydrocarbons.

TABLE 10-6.

REMOVAL MECHANISMS OF TOXIC ORGANICS

Compound	Percept Treatment Achieved		
	Stripping	Sorption	Biological
Nitrogen Compounds			
Acrylonitrile			99.9
Phenols			
Phenol			99.9
2,4-DNP			99.3
2,4-DCP			95.2
PCP		0.58	97.3
Aromatics			
1,2-DCB	21.7		78.2
1,3-DCB			
Nitrobenzene			97.8
Benzene	2.0		97.9
Toluene	5.1	0.02	94.9
Ethylbenzene	5.2	0.19	94.6
Halogenated Hydrocarbons			
Methylene Chloride	8.0	91.7	
1,2-DCE	99.5	0.50	
1,1,1-TCE	100.0		
1,1,2,2,-TCE	93.5		
1,2DCP	99.9		
TCE	65.1	0.83	33.8
Chloroform	19.0	1.19	78.8
Carbon Tetrachloride	33.0	1.38	64.9
Oxygenated Compounds			
Acroleiln			99.9
Polynuclear Aromatics			
Phenanthrene			98.2
Napthalene			98.6
Phthalates			
Bis(2-Ethylhexyl)			76.9
Other			
Ehtyl Acetate	1.0		98.8

Source: Canter and Knox, 1985 as cited by Kincannon and Stover, undated.

Although biological treatment can effectively treat a wide range of organics, it has several drawbacks for hazardous waste site applications. The reliability of the process can be adversely affected by "shock" loads of toxics. Start-up time can be slow if the organisms need to be acclimated to the wastes and the detention time can be long for complex wastes. However, the existence of cultures which have been previously adapted to hazardous wastes can dramatically decrease start-up and detention time.

There are a number of cleanup contractors who have used biological treatment as part of a mobile treatment system. The high purity oxygen treatment process is well suited for mobile treatment applications because the high oxygen efficiency enables use of smaller reactors, shorter detention time, and reduced power consumptions relative to other activated sludge processes. A hazard associated with the high purity oxygen process is that the presence of low flash-point compounds can present a potential fire hazard. However, the system is equipped with hydrocarbon analyzers and control systems that deactivate the system when dangerously high concentrations of volatiles are detected (Ghassemi, Yu, and Quinlivan, 1981). Loss of volatile organics from other biological treatment processes can also pose some localized air pollution and a health hazard to field personnel.

Rotating biological contactors also have advantages for hazardous waste site treatment. The units are compact, and they can handle large flow variations and high organic shock loads, and they do not require use of aeration equipment.

Sludge produced in biological waste treatment may be a hazardous waste itself due to the sorption and concentration of toxic and hazardous compounds contained in the wastewater. If the sludge is hazardous, it must be disposed in a RCRA-approved manner. If the sludge is not hazardous, disposal should conform with State sludge disposal guidelines.

10.1.2.5 Costs

Costs for various sizes of activated sludge units are presented in Table 10-7. The costs for these units assume a detention time of 3 hours, and use of aeration basins, air supply equipment, piping, and a blower building. Clarifier and recycle pumps are not included. The basins are sized to the 50 percent recycle flow. The influent biological oxygen demand (BOD) is assumed to be no greater than 130 ppm and the effluent BOD is assumed to be 40 ppm.

The operation and maintenance costs assume that the hydraulic head loss through the aeration tank is negligible. Sludge wasting and pumping energy are not included.

Union Carbide manufactures a high purity oxygen activated sludge system (UNOX) suitable for mobile system applications. The mobile UNOX systems have a hydraulic capacity of 5 to 40 gpm, are contained within 40 foot van trailers, and include an external clarifier. The oxygen required is also

TABLE 10-7.

GENERAL COST DATA FOR VARIOUS SIZES OF ACTIVATED SLUDGE TREATMENT UNITS

Capacity (gpm)	Construction Costs ($)*	O&M Costs ($/year)*
70	78,500	4,300
140	85,600	6,400
350	107,000	10,000
694	160,000	15,700

*Updated from 1978 to 1984 dollars using third quarter Marshall and Swift Equipment Index.

Source: Adapted from USEPA, 1980.

supplied by Union Carbide. The customer is expected to provide installation labor, operating manpower, analytical support, and utilities. A typical installation requires three to four days (Ghassemi, Yu, and Quinlivan, 1981).

The mobile UNOX system can be either rented or purchased from the Union Carbide Corporation. The estimated rental costs are as follows:

- $6,540 for the checkout and refurbishment of equipment to make it operational
- $550/day for on-site service including engineering consultation on program planning and execution
- $9/day rental of equipment
- Transportation charges to get the equipment from the manufacturer to the site of operation and back again.

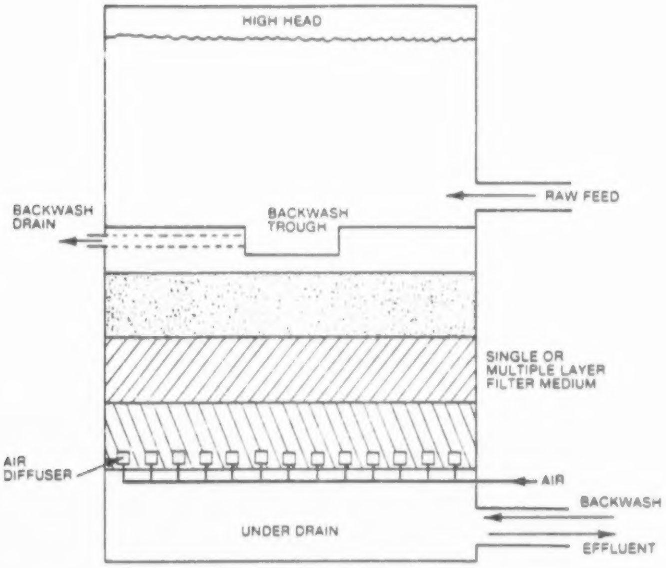
The purchase price for the UNOX mobile unit is between $260,000 and $330,000 (Ghassemi, Yu, and Quinlivan, 1981, updated using 1984 third quarter Marshall Swift Index).

10.1.3 Filtration

10.1.3.1 General Description

Filtration is a physical process whereby suspended solids are removed from solution by forcing the fluid through a porous medium. Granular media filtration is typically used for treating aqueous waste streams. The filter media consists of a bed of granular particles (typically sand or sand with anthracite or coal) (Figure 10-2). The bed is contained within a basin and is supported by an underdrain system which allows the filtered liquid to be drawn off while retaining the filter media in place. As water laden with suspended solids passes through the bed of filter medium, the particles become trapped on top of and within the bed. This either reduces the filtration rate at a constant pressure or increases the amount of pressure needed to force the water through the filter. In order to prevent plugging, the filter is

FIGURE 10-2.
TYPICAL FILTRATION BED



Source: Ghassemi, Yu, and Quinlivan, 1981 backflushed at high velocity to dislodge the particles. The backwash water contains high concentrations of solids and requires further treatment (De Renzo, 1978).

10.1.3.2 Applications/Limitations

Filters find economic application in handling streams containing less than 100 to 200 mg/liter suspended solids, depending on the required effluent level. Increased suspended solids loading will reduce run lengths, and require excessively frequent backwash (De Renzo, 1978). The suspended solids concentration of the filtered liquid depends a great deal on particle size distribution, but typically, granular media filters are capable of producing a filtered liquid with a suspended solids concentration as low as 1 to 10 mg/l. Large flow variations will deleteriously affect effluent quality.

Often, granular media filters are preceded by sedimentation to reduce the suspended solids load on the filter (De Renzo, 1978). Granular media filtration is also frequently installed ahead of biological or activated carbon treatment units to reduce the suspended solids load and in the case of activated carbon to minimize plugging of the carbon columns (De Renzo, 1978).

The granular media filtration process is only marginally effective in treating colloidal size particles. In many cases, these particles can be made larger by flocculation although this will generally reduce run lengths. In cases where it is not possible to flocculate such particles (as in the case of many oil/water emulsions), more advanced techniques such as ultrafiltration may be appropriate (De Renzo, 1978).

10.1.3.3 Design Considerations

The composition and sizing of the filtration bed is an important design consideration. Beds of 4 feet or less composed of 0.5 mm sand and 0.9 mm anthracite are frequently used. However deep-bed filters are also available. It is recommended that pilot plant studies be conducted to determine optimum size and combination of filter material.

A filter bed can function properly only if the backwashing system effectively cleans the material from the filter. Methods which can be used for backwashing include water backwash only, water backwash with auxiliary surface water wash, water wash proceeded by air scour, and simultaneous air and water wash.

The duration of the backwash is about 20 min per cycle. Backwash water, which amounts to 1 to 5 percent of the total flow, can be routed to a primary clarifier often via a storage vessel to allow flow equalization. Several filters are used in parallel to allow continuous processing during backwashing; the backwash cycle usually is automated. Other processes must be sized to handle this recycle flow.

10.1.3.4 Technology/Selection Evaluation

Filtration is a reliable and effective means of removing low levels of solids from wastes provided the solids content does not vary greatly and the filter is backwashed at appropriate intervals.

Filtration equipment is relatively simple, readily available in a wide range of sizes, and easy to operate and control. Filtration is also easily integrated with other treatment steps.

Because of its small space requirements and relatively simple operation, filtration is well suited to mobile treatment systems as well as on-site construction. There is extensive experience with the operation of granular-media filters at hazardous waste sites.

The EPA physical/chemical treatment system which has been in operation for more than 9 years incorporates 3 "dual" media (sand-anthracite) filters connected in parallel in its treatment train. The filters are designed for a maximum hydraulic loading of 7 gpm/ft^2 or 67 gpm (Ghassemi, Yu, and Quinlivan, 1981). There are also a number of manufacturers of package plant systems suitable for being trailer mounted and a number of cleanup contractors who operate mobile treatment systems which include granular media filters as a part of the treatment process.

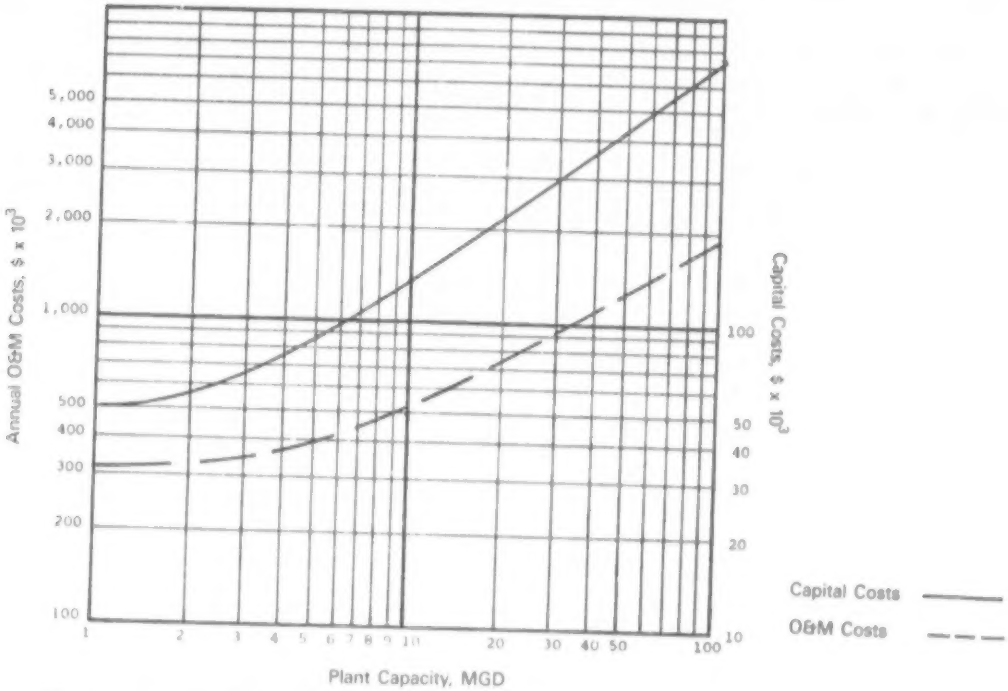
The most obvious maintenance consideration with granular media filtration is handling of the backwash. The backwash will generally contain high concentrations of contaminants and require subsequent treatment.

10.1.3.5 Costs

Figure 10-3 shows construction and operating costs for filtration assuming a filtration rate of 5 gpm/ft^2. A minimimum of 4 filters are assumed to provide flexibility of operation. Capital costs include filter structures, backwash and surface wash systems, media and polymer feeding. Costs of effluent filtering and pumping are not included. Power costs are based on each filter backwashing once per 12 hours (Culp, Wesner and Culp, 1978).

The construction costs assume a filtration rate of 2 gpm/square foot and 76 in filter media (silica sand and anthracite coal mixture) depths inside two open-topped cylindrical steel tanks. The construction costs also include chemical feed systems (alum, soda ash, polymer, and chlorine), pumps, pre-filter contact basin, a backwash storage basin, building, and all necessary piping. The operation and maintenance costs include all building utilities, process utilities, routine maintenance costs, and replacement of filter media lost through normal backwash operations (Hansen, Gumerman, and Culp, 1979). The operation and maintenance costs do not include treatment chemicals because usage rates of these chemicals would vary considerably

FIGURE 10-3. COST OF EFFLUENT FILTRATION*



***Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.908)**

Source: Culp, Wesner & Culp, 1978

depending on the wastestream characteristics. Unit costs for treatment chemicals are presented in Section 9.2. Costs for various sizes of package neutralization, precipitation, and filtration plants are presented in Table 10-8.

10.1.4 Precipitation/Flocculation

10.1.4.1 General Description

Precipitation is a physiochemical process whereby some or all of a substance in solution is transformed into a solid phase. It is based on alteration of the chemical equilibrium relationships affecting the solubility of inorganic species. Removal of metals as hydroxides or sulfides is the most common precipitation application in wastewater treatment. Generally, lime or sodium sulfide is added to the wastewater in a rapid mixing tank along with flocculating agents (described below). The wastewater flows to a flocculation chamber in which adequate mixing and retention time is provided for agglomeration of precipitate particles. Agglomerated particles are separated from the liquid phase by settling in a sedimentation chamber, and/or by other

TABLE 10-8.

GENERAL COST DATA FOR VARIOUS SIZES OF NEUTRALIZATION, PRECIPITATION, AND FILTRATION UNITS

Plant Capacity (gpm)	Construction Cost ($)*	Operation and Maintenance ($/year)*
4	78,770	20,730
8	89,610	21,390
40	126,520	26,550
80	179,300	47,960
140	253,610	56,700
225	293,670	57,560
280	396,960	61,860
560	619,940	92,000

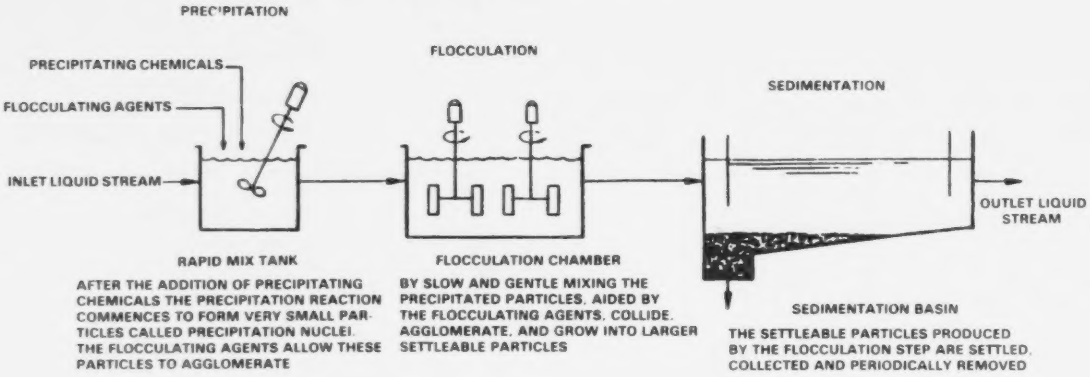
*Updated from 1979 to 1984 dollars using third quarter Marshall and Swift Index.

Source: Adapted from Hansen, Gumerman, and Culp, 1979.

physical processes such as filtration. Figure 10-4 illustrates a typical configuration for precipitation flocculation and sedimentation.

Although precipitation of metals is governed by the solubility product of ionic species, in actual practice, effluent concentrations equal to the solubility product are rarely achieved. Usually, the amount of lime added is about three times the stoichiometric amount that would be added to reduce solubility due to the common ion effect. Figure 10-5 gives solubilities of various metal hydroxides and sulfides at various pH levels. The metal sulfides have significantly lower solubility than their hydroxide counterparts and more complete precipitation is achieved. Metal sulfides are also stable over a broad pH range. Many metal hydroxides, on the other hand, are stable only over a narrow pH range; metals reach a minimum solubility at a specific pH, but further addition of lime causes the metal to become soluble again. Therefore, dosages of lime need to be accurately controlled. This may be particularly challenging when working with aqueous wastes from waste disposal sites where wide variations in flow rates and quantities of metals are to be expected. The stabilities of metal carbonates are also quite dependent on pH.

FIGURE 10-4.
REPRESENTATIVE CONFIGURATION EMPLOYING PRECIPITATION, FLOCCULATION, AND SEDIMENTATION


PRECIPITATION
PRECIPITATING CHEMICALS
FLOCCULATING AGENTS
INLET LIQUID STREAM
FLOCCULATION
SEDIMENTATION
OUTLET LIQUID STREAM
RAPID MIX TANK
AFTER THE ADDITION OF PRECIPITATING CHEMICALS THE PRECIPITATION REACTION COMMENCES TO FORM VERY SMALL PARTICLES CALLED PRECIPITATION NUCLEI. THE FLOCCULATING AGENTS ALLOW THESE PARTICLES TO AGGLOMERATE
FLOCCULATION CHAMBER
BY SLOW AND GENTLE MIXING THE PRECIPITATED PARTICLES, AIDED BY THE FLOCCULATING AGENTS, COLLIDE, AGGLOMERATE, AND GROW INTO LARGER SETTLEABLE PARTICLES
SEDIMENTATION BASIN
THE SETTLEABLE PARTICLES PRODUCED BY THE FLOCCULATION STEP ARE SETTLED, COLLECTED AND PERIODICALLY REMOVED

Source: De Renzo, 1978

Flocculation is used to describe the process by which small, unsettleable particles suspended in a liquid medium are made to agglomerate into larger, more settleable particles. The mechanisms by which flocculation occurs involve surface chemistry and particle change phenomena. In simple terms, these various phenomena can be grouped into two sequential mechanisms (Kiang and Metry, 1982):

- chemically induced destabilization of the requisite surface-related forces, thus allowing particles to stick together when they touch and

- chemical bridging and physical enmeshment between the now nonrepelling particles, allowing for the formation of large particles.

Flocculation involves three basic steps:

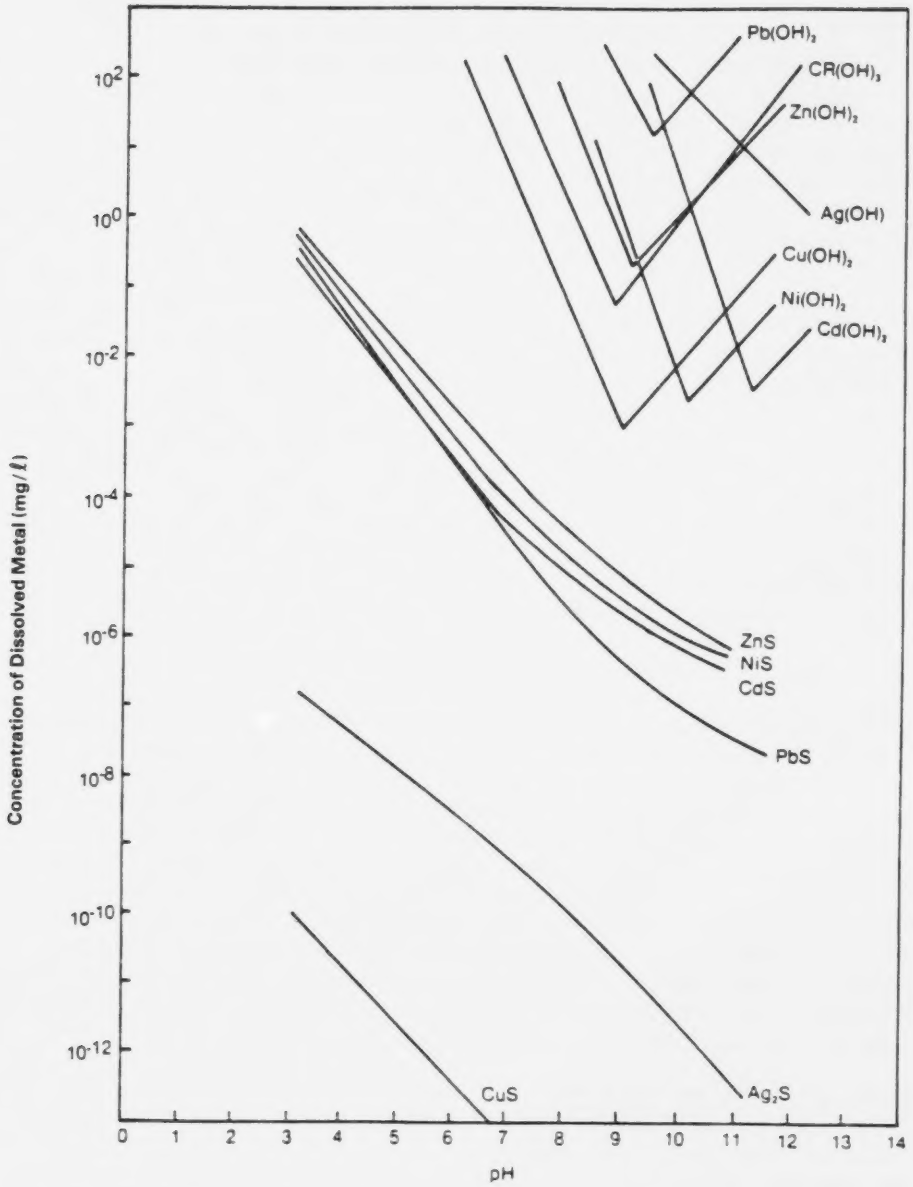
- addition of flocculating agent to the waste stream

- rapid mixing to disperse the flocculating agent

- slow and gently mixing to allow for contact between small particles.

Typically, chemicals used to cause flocculation include alum, lime, various iron salts (ferric chloride, ferrous sulfate) and organic flocculating agents, often referred to as "polyelectrolytes." These materials generally consist of long-chain, water-soluble polymers such as polyacrylamides. They are used either in conjunction with the inorganic flocculants, such as alum, or as the primary flocculating agent. A polyelectrole may be termed cationic, anionic or ampholytic depending upon the type of ionizable groups; or nonionic

FIGURE 10-5.
SOLUBILITY OF METAL HYDROXIDES AND SULFIDES



Source: Ghassemi, Yu, and Quinlivan, 1981 if it contains no ionizable groups. The range of physical/chemical characteristics (e.g., density, viscosity, toxicity and molecular weight) of the several thousand available polymers is extremely broad.

The inorganic flocculants, such as alum, lime or iron salts, make use of precipitation reactions. Alum (hydrated aluminum sulfate) is typically added to aqueous waste streams as a solution. Upon mixing, the slightly higher pH of the water causes the alum to hydrolyze and form fluffy, gelatinous precipitates of aluminum hydroxide. These precipitates, partially due to their large surface area, enmesh small particles and thereby create larger particles. Lime and iron salts also have a tendency to form large fluffy precipitates or "floc" particles. Many precipitation reactions, such as the precipitation of metals from solution by the addition of sulfide ions, do not readily form floc particles, but rather precipitate as very fine and relatively stable colloidal particles. In such cases, flocculating agents such as alum and/or polyelectrolytes must be added to cause flocculation of the metal sulfide precipitates (Canter and Knox, 1985).

Once suspended particles have been flocculated into larger particles, they usually can be removed from the liquid by sedimentation, provided that a sufficient density difference exists between the suspended matter and the liquid.

10.1.4.2 Applications/Limitations

Precipitation is applicable to the removal of most metals from wastewater including zinc, cadmium, chromium, copper, fluoride, lead, manganese, and mercury. Also, certain anionic species can be removed by precipitation, such as phosphate, sulfate, and fluoride.

Precipitation is useful for most aqueous hazardous waste aqueous streams. However, limitations may be imposed by certain physical or chemical characteristics. In some cases, organic compounds may form organometallic complexes with metals, which could inhibit precipitation. Cyanide and other ions in the wastewater may also complex with metals, making treatment by precipitation less efficient.

Flocculation is applicable to any aqueous waste stream where particles must be agglomerated into larger more settleable particles prior to sedimentation or other types of treatment. There is no concentration limit for precipitation or flocculation. Highly viscous waste streams will inhibit settling of solids.

In addition to being used to treat waste streams, precipitation can also be used as an in-situ process to treat aqueous wastes in surface impoundments. In an in-situ application, lime and flocculants are added directly to the lagoon, and mixing, flocculation, and sedimentation are allowed to occur within the lagoon. In some cases, wind and pumping action can provide the energy for mixing.

10.1.4.3 Design Considerations

Selection of the most suitable precipitate or flocculant and their optimum dosages is determined through laboratory jar test studies. In addition to determining the appropriate chemicals and optimum chemical dosages, other important parameters which need to be determined as part of the overall design include (Canter and Knox, 1985):

- most suitable chemical addition system
- optimum pH requirement
- rapid mix requirements
- sludge production
- sludge flocculation, settling and dewatering characteristics.

10.1.4.4 Technology Selection/Evaluation

Precipitation and flocculation are well established technologies and the operating parameters are well defined. The processes requires only chemical pumps, metering devices, and mixing and settling tanks. The equipment is readily available and easy to operate. Precipitation and flocculation can be easily integrated into more complex treatment systems.

The performance and reliability of precipitation and flocculation depends greatly on the variability of the composition of the waste being treated. Chemical addition must be determined using laboratory tests and must be adjusted with compositional changes of the waste being treated or poor performance will result.

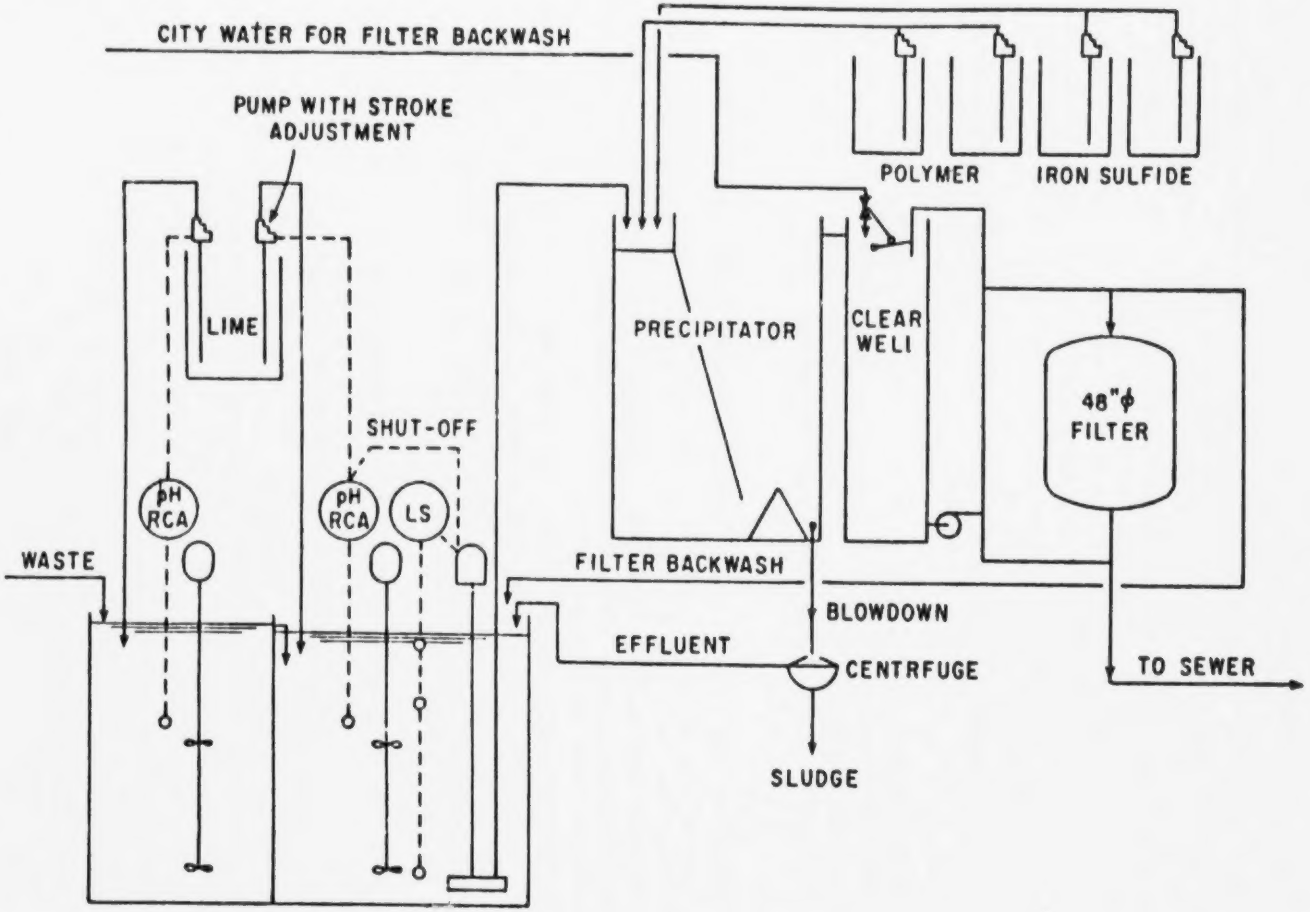
Precipitation is nonselective in that compounds other than those targeted may be removed. Both precipitation and flocculation are nondestructive and generates a large volume of sludge which must be disposed.

Precipitation and flocculation poses minimal safety and health hazards to field workers. The entire system is operated at near ambient conditions, eliminating the danger of high pressure/high temperature operation with other systems. While the chemicals employed are often skin irritants, they can easily be handled in a safe manner.

10.1.4.5 Costs

Table 10-9 shows a breakdown of costs for the 40 gpm sulfex heavy metal removal system illustrated in Figure 10-6.

FIGURE 10-6. 40 GPM SULFEX PLANT FOR COMBINED REMOVAL OF Cr^{+6},Zn,Cu,Cd,Ni and Fe.



Source: Metal Finisher's Foundation, 1977

TABLE 10-9. 1985 CAPITAL COSTS* FOR SULFEX HEAVY METAL PRECIPITATION SYSTEM

Selling Price	
Equipment	Price
1. Precipitator with clear well, centrifuge for dewatering,chemical feeds and agitators, and engineering drawings	$68,768
2. Filter with transfer pump and engineering drawings	7,623
3. Neutralization systsem including agitators, chemical feeds, pH controls, sump pump, and engineering drawings	86,126
Total selling price	$162,517
Installation Cost (estimated by outside contractor)	$66,657
TOTAL	$229,174

*Costs were updated to $1985 using the 1983 and 1985 ENR Construction Cost Index.

Source: Metal Finisher's Foundation, 1977

The precipitator is sized to operate at a surface rate of 1.6 gpm/ft^2 and the filter at a surface rate of 3.2 gpm/ft^2. Chemical costs for the Sulfex process and a hydroxide precipitation process are shown in Table 10-10. These costs were estimated for treament of an influent containing 4 mg/l Cu, Cd, Cr^{+3}, Ni, and Zn at pH 6.0.

Figure 10-7 shows capital and operating costs for a flocculation system including chemical storage, chemical feeding and rapid mix. A polymer dosage of 1 mg/l at 0.25 percent solution is assumed. Construction costs also include piping and building to house the feeding equipment and bag storage.

Construction costs include: Piping and building to house the feeding equipment and bag storage. 1 Mgal/d plant size and smaller use manual feed procedures. 2 systems of tanks and feeders are included. 10 Mgal/d plant size includes cost of feeders and mixing tanks, one day tank and 2 solution feeders. 100 Mgal/d plant size includes costs for 4 feeders and mixing tanks, 2 holding tanks and 10 solution feeders. The rapid mix tank is concrete,

TABLE 10-10. COMPARISON OF CHEMICAL COSTS* OF HYDROXIDE AND SULFIDE PRECIPITATION PROCESSES

(1) Hydroxide Process

	Eff. Qyal. (mg/l)		Chemical	Dosage lb/1000 gal.		Cost		
							¢/1000 gal.	
	pH 7.5	pH 10		pH 7.5	pH 10	¢/lb	pH 7.5	pH 10
Cu	0.1	<0.1	$CA(OH)_2$	0.33	0.92	3.16	1.05	2.99
Cd	3.8	<0.1	Polymer	0.03	0.03	105.4	3.16	4.22
Cr	<0.5	<0.1	H_2SO_4	-	0.61	8.8	-	5.45
Ni	2.3	<0.1						
Zn	1.3	<0.1				Total	4.21	12.66

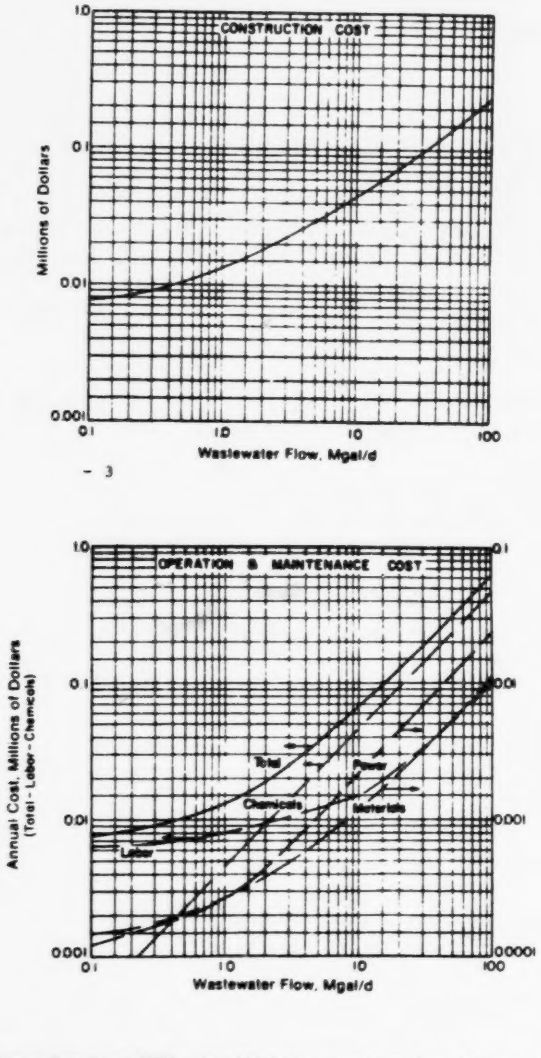
(2) Sulfex Process

	Eff. Qual. (mg/l)	Chemical	Dosage lb/1000 gal.	Cost	
					/1000 gal.
	pH 8.5		pH 8.5	.lb	pH 8.5
Cu	0.01	71% NaHS	0.09	19.77	1.76
Cd	0.1	$FeSO_4.7H_2O$	0.77	3.95	2.99
Cr	<0.05	Polymer	0.03	105.4	3.16
Ni	0.05	$Ca(OH)_2$	1.13	3.16	3.51
Zn	0.01				
				Total	11.42

*Costs were updated to $1985 using the 1977 and 1985 ENR Construction Cost Index.

Source: Metal Finisher's Foundation, 1977

FIGURE 10-7. POLYMER ADDITION COSTS*



*Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.303)

Source: USEPA, 1982a

equipped with stainless steel mixer and handrails. 0.1 Mgal/d plant size: no separate building is required. Manual operation of feeder, mix tank solution feeder and holding tank.

10.1.5 Sedimentation Technology

10.1.5.1 General Description

Sedimentation is a process that relies upon gravity to remove suspended solids in an aqueous waste stream. The fundamentals of a sedimentation process includes (Kiang and Metry, 1982):

- A basin or container of sufficient size to maintain the liquid to be treated in a relatively quiescent state for a specified period of time
- A means of directing the liquid to be treated into the above basin in a manner conducive to settling
- A means of physically removing the settled particles from the liquid (or liquid from the settled particles).

Sedimentation can be carried out as either a batch or continuous process in lined impoundments, conventional settling basins, clarifiers, and high rate gravity settlers. Modified aboveground swimming pools have been used many times for sedimentation in temporary, short-term treatment systems at hazardous waste sites. Figure 10-8 illustrates three different design configurations for sedimentation. In sedimentation ponds the liquid is merely decanted as the particles accumulate on the bottom of the pond. Backhoes, draglines, or siphons are used periodically to remove settled solids.

Sedimentation basins and clarifiers usually employ a built-in solids collection and removal devices such as a sludge scrapper and draw-off mechanism. Sedimentation basins are general rectaggular, usually employ a belt-like collection mechanism, and are mainly used for removal of truly settleable particles from liquid.

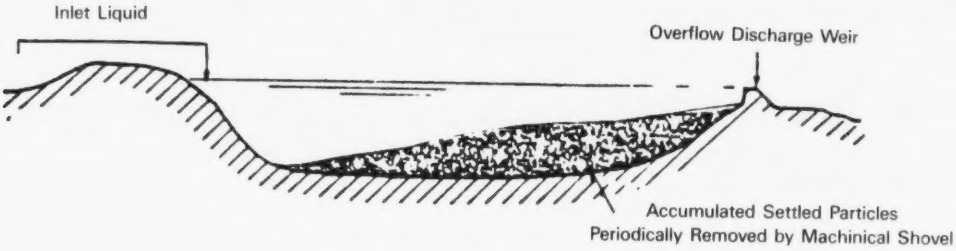
Clarifiers are usually circular and are used in applications involving precipitation and flocculation as well as sedimentation. Many clarifiers are equipped with separate zones for chemical mixing and precipitation, floculation, and sedimentation (Kiang and Metry, 1982).

10.1.5.2 Applications/Limitations

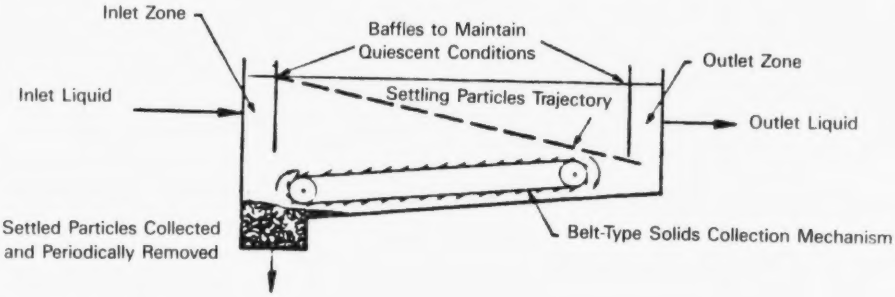
Sedimentation is commonly applied to aqueous wastes with high suspended solid loadings. This may include surface run-off, collected leachate or landfill toe seepage, dredge slurries, and effluents from biological treatment and precipitation/flocculation. Sedimentation is also required as a

FIGURE 10-8.
REPRESENTATIVE TYPES OF SEDIMENTATION

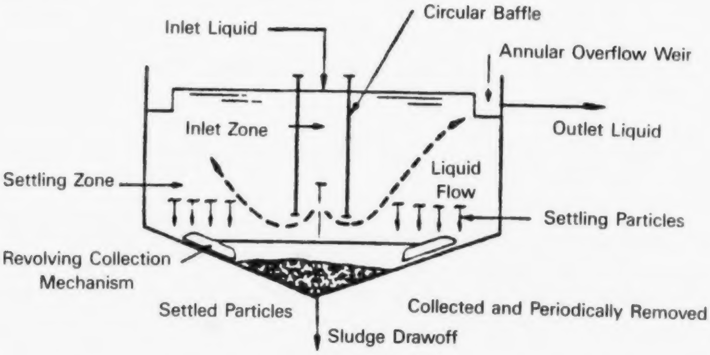
Setting Pond


Inlet Liquid
Overflow Discharge Weir
Accumulated Settled Particles
Periodically Removed by Machinical Shovel

Sedimentation Basin



Circular Clarifier



Source: De Renzo, 1978

pretreatment step for many chemical processes, including carbon adsorption, ion exchange, stripping, reverse osmosis and filtration.

This technology is applicable to the removal of suspended solids heavier than water. Suspended oil droplets or oil-soaked particles may not settle out and may have to be removed by some other means. Some sedimentation units are fitted with skimmers to remove oil and grease that float to the water surface. However, these would not be effective in removing emulsified oils.

10.1.5.3 Design Consideration

Sedimentation is frequently considered in terms of ideal setting. The ideal setting theory results in the following equation for surface loading or overflow rate.

$$V_o = \frac{Q}{A}$$

where: V_o = setting velocity
Q = flow through the basin
A = surface area of the basin

Sedimentation basin loadings (Q/A) are often expressed in terms of gallons per day per square foot. Thus under ideal settling conditions, sedimentation is independent of basin depth and detention time, and depends only on the flow rate, basin surface area and properties of the particle.

However sedimentation does not perform according to ideal settling conditions since settling is affected by such conditions as turbulence, and bottom scour. Therefore removal of particles is dependent on basin depth, and detention time as well as flow rate surface area and particle size. The performance of a sedimentation basin on a suspension of discrete particles can be calculated, but it is not possible to calculate sedimentation basin performance for a suspension of flocculating particles, such as a wastewater, because settling velocities change continually. Laboratory settling tests, however, may be performed to predict sedimentation basin performance.

10.1.5.4 Technology Selection/Evaluation

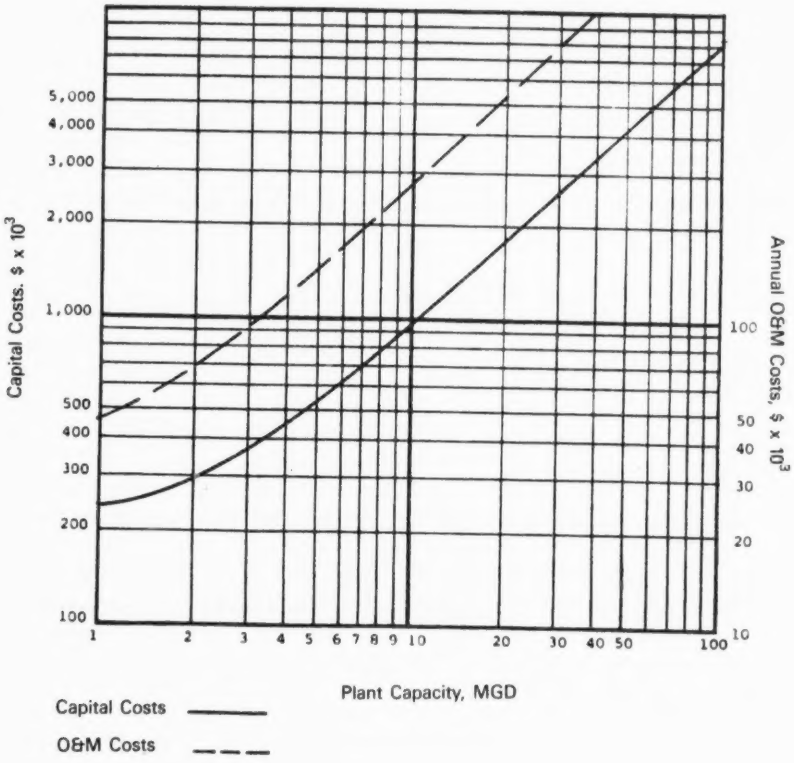
Sedimentation provides a reliable means to remove suspended matter from a waste stream, provided the suspended matter is settleable and the treatment process including the use of flocclants/coagulants has been appropriately designed from laboratory settling tests. Most clarifiers are capable of removing 90 to 99 percent of the suspended solids.

Sedimentation employs readily available equipment and is relatively easy to operate. The process is versatile in that it can be applied to almost any liquid waste stream containing suspended solids. It can also be easily integrated into a more complex treatment system as a pre- or post-treatment method. Sedimentation is nonselective and nondestructive, resulting in a large volume of potentially contaminated sludge that may require further treatment and disposal.

10.1.5.5 Costs

The cost of a system which includes chemical clarification, rapid mixing, flocculation with alum and polymer and sedimentation is shown in Figure 10-9. The cost estimate assumed alum and polymer dosages of 200 mg/l and 1 mg/l respectively, and a flow rate to the clarifier of 800 gpd/ft^2. The costs of chemical sludge proceessing and disposal are not included in the capital costs. O&M costs include cost of chemical purchase (Culp, Wesner and Culp, 1978).

FIGURE 10-9. COST OF CHEMICAL CLARIFICATION WITH ALUM*



*Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.908)

Source: Culp, Wesner & Culp, 1978

10.1.6 Ion Exchange and Sorptive Resins

10.1.6.1 General Description

Ion exchange is a process whereby the toxic ions are removed from the aqueous phase by being exchanged with relatively harmless ions held by the ion exchange material. Modern ion exchange resins are primarily synthetic organic materials containing ionic functional groups to which exchangeable ions are attached. These synthetic resins are structurally stable (that is, can tolerate a range of temperature and pH conditions), exhibit a high exchange capacity, and can be tailored to show selectivity towards specific ions. Exchangers with negatively-charged sites are cation exchangers because they take up positively charged ions. Anion exchangers have positively charged sites and, consequently, take up negative ions. The exchange reaction is reversible and concentration dependent, and it is possible to regenerate the exchange resins for reuse. Sorptive (macroporous) resins are also available for removal of organics and the removal mechanism is one of sorption rather than ion exchange (Ghassemi, Yu, and Quinlivan, 1981).

10.1.6.2 Applications/Limitations

Ion exchange is used to remove a broad range of ionic species from water including:

- All metallic elements when present as soluble species, either anionic or cationic
- Inorganic anions such as halides, sulfates, nitrates, cyanides, etc.
- Organic acids such as carboxylics, sulfonics, and some phenols, at a pH sufficiently alkaline to give the ions
- Organic amines when the solution acidity is sufficiently acid to form the corresponding acid salt (De Renzo, 1978).

Sorptive resins can remove a wide range of polar and non-polar organics.

A practical upper concentration limit for ion exchange is about 2,500 to 4,000 mg/l. A higher concentration results in rapid exhaustion of the resin and inordinately high regeneration costs. Suspended solids in the feed stream should be less than 50 mg/l to prevent plugging the resins, and waste streams must be free of oxidants (De Renzo, 1978).

10.1.6.3 Design Considerations

Specific ion exchange and sorptive resins systems must be designed on a case-by-case basis. It is useful to note that although there are three major operating models (fixed bed cocurrent, fixed bed countercurrent, and continuous countercurrent), fixed bed countercurrent systems are most widely used. Figure 10-10 illustrates the fixed bed countercurrent and continuous countercurrent systems. The continuous countercurrent system is suitable for high flows. Complete removal of cations and anions ("demineralization") can be accomplished by using the hydrogen form of a cation exchange resin and the hydroxide form of an anion exchange resin. For removal of organics as well as inorganics, a combination adsorptive/demineralization system, can be used. In this system, lead beds would carry sorptive resins which would act as organic scavengers, and the end beds would contain anion and cation exchange resins. By carrying different types of adsorptive resins (e.g., polar and non-polar), a broad spectrum of organics could be removed (Ghassemi, Yu, and Quinlivan, 1981).

10.1.6.4 Technology Selection/Evaluation

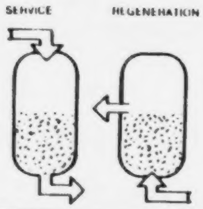
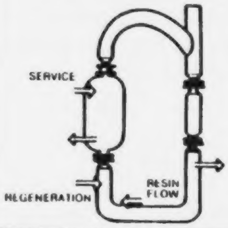
Ion exchange is a well established technology for removal of heavy metals and hazardous anions from dilute solutions. Ion exchange can be expected to perform well for these applications when fed wastes of variable composition, provided the system's effluent is continually monitored to determine when resin bed exhaustion has occurred. However, as mentioned previously, the reliability of ion exchange is markedly affected by the presence of suspended solids. Use of sorptive resins is relatively new and reliability under various conditions is not as well known.

Ion exchange systems are commercially available from a number of vendors. The units are relatively compact and are not energy intensive. Start-up or shut-down can be accomplished easily and quickly (Ghassemi, Yu, and Quinlivan, 1981). These features allow for convenient use of ion exchange and sorptive resin systems in mobile treatment systems.

Although exchange columns can be operated manually or automatically, manual operation is better suited for hazardous waste site applications because of the diversity of wastes encountered; with manual operation, the operator can decide when to stop the service cycle and begin the backwash cycle. However, this requires use of a skilled operator familiar with the process (Ghassemi, Yu, and Quinlivan, 1981).

Use of several exchange columns at a site can provide considerable flexibility. As described previously, various resin types can be used to remove anions, cations, and organics. Various columns can be arranged in series to increase service-life between regeneratation of the lead bed or in parallel for maximum hydraulic capacity. The piping arrangement would allow for one or more beds to be taken out for regeneration while the remaining columns would remain in service. (Ghassemi, Yu, Quinlivan, 1981).

FIGURE 10-10.
PERTINENT FEATURES OF ION EXCHANGE SYSTEMS

Types	Countercurrent Fixed Bed  	Continuous Countercurrent 
Description of Process	Regeneration flows opposite in direction to influent. Backwash (in regeneration) does not occur on every cycle to preserve resin stage heights. Resin bed is locked in place during regeneration.	Multi-stage countercurrent movement of resin in closed loop providing simultaneous treatment, regeneration, backwash and rinse. Operation is only interrupted for momentary resin pulse.
Indications for Use	Handles high loads at moderate thruput or low loads at high thruput (GPM x TDS or GPM x PPM removal = 40,000 or more). Where effluent quality must be relatively constant, regeneration cost is relatively critical, disposal of single batch waste volume no problem.	Highloads with high thruputs (GPM x TDS or GPM x PPM removal = 40,000 or more). Where constant effluent quality is essential, regeneration costs critical, total waste volume requires small, concentrated stream to be controllable. Where loss of product thru dilution and waste must be minimized. Where available floor space is limited
Advantages	Moderate capital cost. Can be operated with periodic attention. Moderate regeneration cost. Lesser volume of waste due to less frequent backwash. Consistent effluent quality	Lowest regeneration cost. Lowest resin inventory. Consistent effluent quality. Highest thruput to floor space. Large capacity units factory preassembled. Concentrated low-volume waste stream. Can handle strong chemical solutions and slurry. Fully automatic operation
Disadvantages	Increased controls and instrumentation, higher cost. Requires mechanism to lock resin bed. Large single batches of waste disposal. Moderate water consmption thru dilution and waste. Requires substantial floor space	Requires automatic controls and instrumentation, higher capital cost. More headroom required

Source: Chemical Seperations Corporation

Consideration must be given to disposal of contaminated ion exchange regeneration solution. In addition to proper disposal, another important operational consideration is the selection of regeneration chemicals. Caution must be exercised in making this selection to ensure the compatibility of the regenerating chemical with the waste being treated. For example, the use of nitric acid to regenerate an ion exchange column containing ammonium ions results in the formation of ammonium nitrate, a potentially explosive compound.

10.1.6.5 Costs

Costs for various sizes of ion exchange units are presented in Table 10-11. The construction costs assume fabricated steel contact vessels with baked phenolic linings, a resin depth of 6 feet, housing for the columns, and all piping and backwash facilities.

Operation and maintenance costs include electricity for backwashing (after 150 bed volumes have been treated) and periodic repair and replacement costs. Costs for regenerant chemicals are not included because they vary depending on the types and concentrations of target chemicals to be removed from the wastewater.

TABLE 10-11.

GENERAL COST DATA FOR VARIOUS SIZES OF EXCHANGE UNITS

Plant Capacity (gpm)	Construction Cost ($)*	Operation and Maintenance Costs ($/year)*
50	84,105	14,530
195	116,200	21,260
305	134,770	24,280
438	154,000	27,590
597	180,270	31,531

*Updated from 1979 to 1984 dollars using third quarter Marshall and Swift Index.

Source: Adapted from Hansen, Gumerman, and Culp, 1979.

10.1.7 Reverse Osmosis

10.1.7.1 General Description

Osmosis is the spontaneous flow of solvent (e.g., water) from a dilute solution through a semipermeable membrane (impurities or solute permeates at a much slower rate) to a more concentrated solution. Reverse osmosis is the application of sufficient pressure to the concentrated solution to overcome the osmotic pressure and force the net flow of water through the membrane toward the dilute phase. This allows the concentration of solute (impurities) to be built up in a circulating system on one side of the membrane while relatively pure water is transported through the membrane. Ions and small molecules in true solution can be separated from water by this technique.

The basic components of a reverse osmosis unit are the membrane, a membrane support structure, a containing vessel, and a high pressure pump. The membrane and membrane support structure are the most critical elements.

10.1.7.2 Applications/Limitations

Reverse osmosis (RO) is used to reduce the concentrations of dissolved solids, both organic and inorganic. In treatment of hazardous waste contaminated streams, use of reverse osmosis would be primarily limited to polishing low flow streams containing highly toxic contaminants. In general, good removal can be expected for high molecular weight organics and charged anions and cations. Multivalent ions are treated more effectively than are univalent ions. Recent advances in membrane technology have made it possible to remove such low moelcular weight organics as alcohols, ketones, amines, and aldehydes (Gooding, 1985). Table 10-12 shows removal results obtained during testing of a mobile RO unit using two favorable membrane materials (Whittaker, 1984).

RO Units are subject to chemical attack, fouling, and plugging. Pretreatment requirements can be extensive. Wastewater must be pretreated to remove oxidizing materials such as iron and maganese salts, to filter out particulates, adjust pH to a range of 4.0 to 7.5, and to remove oil, grease, and other film forms (De Renzo, 1978). The growth of slimy biomass on the membrane surface or the presence of organic macromolecules may also foul the membrane. This organic fouling can be minimized by prechlorination, addition of biocides and/or pretreatment with activated carbon (Ghassemi, Yu, and Quinlivan, 1981).

10.1.7.3 Design Considerations

The most critical design consideration applicable to reverse osmosis technology is the design of the semipermeable membrane. In addition to

TABLE 10-12.

RESULTS OF PILOT SCALE TESTING OF A REVERSE OSMOSIS UNIT

			Percent removed in permeate	
Chemical	Feed Concentration (ppb)	Percent Concentrated in Concentrate	Polyether-polysulphone membrane	Polyester/amide poly-sulphone membrane
Dichloromethane	406	203	58	52
Acetone	110	355	84	76
1,1-Dichloroethene	34	795	99	95
Tetrahydrofuran	17,890	467	98	89
Diethyl ether	210	439	97	89
Chloroform	270[1]	567	98	92
1,2-Dichlorethane	99	415	92	85
1,1,1-Trichloroethane	659	651	99.8	97
Trichloroethene	24[1]	346	99	92
Benzene	539	491	99	99
Bromoform	12[1]	633	99.1	98
Hexane	10[1]	704	99.8	97

[1] no standard available; concentration estimated.

Source: Whittaker, 1984

allowing the achievement of the required degree of separation at an economic flux level under ideal conditions, the membrane must be incorporated in an operating system which satisfies these practical requirements (Conway and Ross, 1980):

- Minimum concentration polarization, ie., ratio of impurity concentration at the membrane surface to that in the bulk stream
- High packing density, i.e., membrane surface area per unit volume of the pressure module
- Ability to handle any particulate impurities (by proliferation if necessary)
- Adequate support for the membrane and other physical features such as effectiveness of seals, ease of membrane replacement, and ease of cleanning.

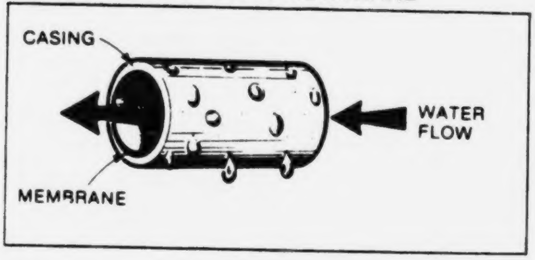
Membranes are usually fabricated in flat sheets or tubular forms and are assembled into modules. The most common materials used are cellulose acetate and other polymers such as polyamides and polyether-polysulphone. There are three basic module designs: tubular, hollow fiber, and spiral wound. These are illustrated in Figure 10-11. Each type of membrane module has its own advantages and limitations. The tubular module provides the largest flow channel and allows for turbulent fluid flow regime; thus, it is least susceptible to plugging caused by suspended solids and has the highest flux. However, because of its small area/volume ratio the total product recovered per module is small. The cost of a tubular module is approximately five times that for the other modules for an equivalent rate of water recovery, and the total space requirement is about three to five times that for the spiral wound system (Ghassemi, Yu and Quinlivan, 1981).

A hollow fiber membrane is constructed of polyamide polymers and cellulose triacetate by Dupont and Dow, respectively. The polyamide membrane permits a wider operating pH range than cellulose acetate, which is commonly used for the construction of spiral wound and tubular membranes. The flow channel and the flux are about an order of magnitude lower than thee other configurations. This small flux, however, is compensated for by the large surface area/volume ratio, with the total product water per module being close to that obtainable with spiral wound modules. However, because of the small size of the channels (about 0.004 in.) and the laminar fluid flow regime within the channels, this module is susceptible to plugging and may require extensive pretreatment to protect the membrane (Ghassemi, Yu and Quinlivan, 1981).

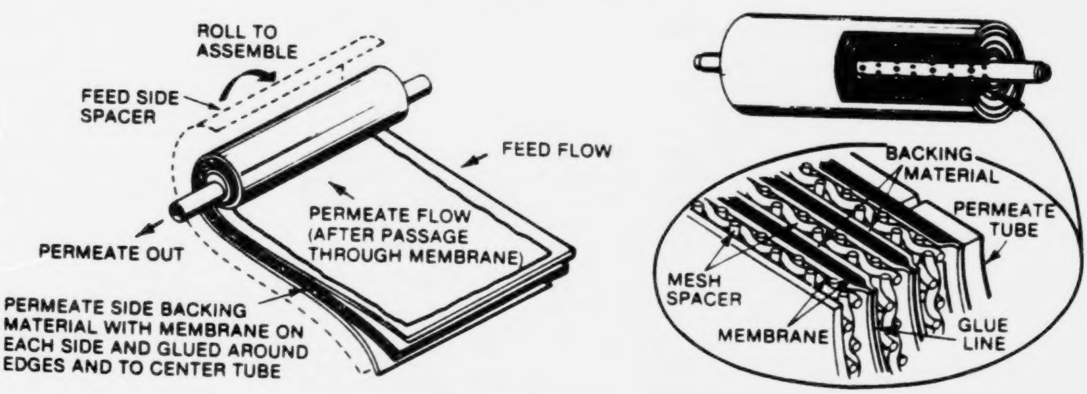
The spiral wound module consists of an envelope of flat sheet membranes rolled around a permeate collector tube. This configuration provides for a higher flux and greater resistance to fouling than the hollow fiber modules; it is also less expensive and occupies less space than a tubular module (Ghassemi, Yu and Quinlivan, 1981).

FIGURE 10-11. MEMBRANE MODULE CONFIGURATIONS

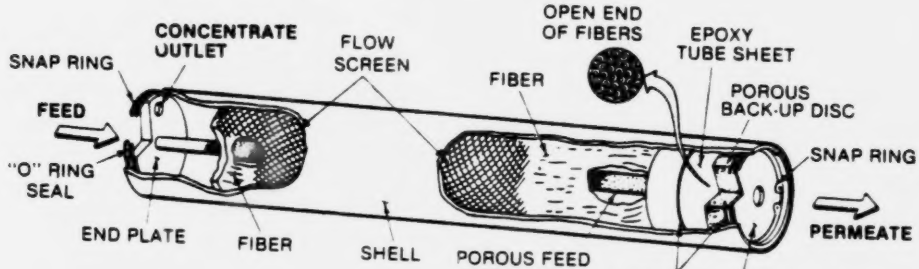
A. TUBULAR MEMBRANE



b. SPIRAL-WOUND MODULE


ROLL TO ASSEMBLE
FEED SIDE SPACER
FEED FLOW
PERMEATE FLOW (AFTER PASSAGE THROUGH MEMBRANE)
PERMEATE OUT
PERMEATE SIDE BACKING MATERIAL WITH MEMBRANE ON EACH SIDE AND GLUED AROUND EDGES AND TO CENTER TUBE
BACKING MATERIAL
PERMEATE TUBE
MESH SPACER
MEMBRANE
GLUE LINE

c. HOLLOW-FIBER MODULE



Source: Ghassemi, Yu & Quinlivan, 1981

10.1.7.4 Technology Selection/Evaluation

Reverse osmosis is an effective treatment technology for removal of dissolved solids presuming appropriate pretreatment has been performed for suspended solids removal, pH adjustments, and removal of oxidizers, oil, and grease. Because the process is so susceptible to fouling and plugging, on-line monitors may be required to monitor pH, suspended solids, etc. on a continuous basis. Reverse osmosis has not been widely used for treatment of hazardous wastes.

Reverse osmosis will not reliably treat wastes with a high organic content, as the membrane may dissolve in the waste. Lower levels of organic compounds may also be detrimental to the unit's reliability, as biological growth may form on a membrane fed an influent containing biodegradable organics.

The fact that RO units can be operated in series or in parallel provides some flexibility in dealing with increased flow rates or concentration of dissolved species.

Memtek Corporation of Ontario, Canada has developed a mobile reverse osmosis unit for Environment Canada. The unit, which is capable of handling low flows of about 10 gpm is currently being tested for various types of spills (Whittaker, 1984).

The volume of the reject generated by reverse osmosis is about 10 to 25 percent of the feed volume. Provisions must be made to treat this potentially hazardous waste.

10.1.7.5 Costs

Costs for various sizes of reverse osmosis units are presented in Table 10-13. The construction costs include housing, tanks, piping, membranes, flow meters, cartridge filters, acid and polyphosphate feed equipment, and cleanup equipment. These costs are based on influent total dissolved solids concentrations of less than 10,000 ppm.

The operation and maintenance costs include electricity for the high pressure feed pumps (450 psi operating pressure), building utilities, routine periodic repair, routine cleaning, and membrane replacement every 3 years. Operation and maintenance costs do not include costs for pretreatment chemicals due to extreme usage rate variability between plants.

TABLE 10-13.

GENERAL COST DATA FOR VARIOUS SIZES OF REVERSE OSMOSIS UNITS

Plant Capacity (gpm)	Construction Costs ($)*	O&M Costs ($/year)
1.74	17,070	7,580
7	33,280	12,070
70	171,820	40,829
700	1,014,600	249,930

*Updated from 1979 to 1984 dollars using third quarter Marshall and Swift Index.

Source: Adapted from Hansen, Gumerman, and Culp, 1979.

The RO unit being tested by Environment Canada can handle flow up to 11,000 gpd and costs approximately $60,000. Membranes vary considerably in cost. The Toray and DSI membranes discussed in Table 10-12, for example, cost $360 and $915, respectively (Whittaker et al., 1985).

10.1.8 Neutralization

10.1.8.1 General Description

Neutralization consists of adding acid or base to a waste in order to adjust its pH. The most common system for neutralizing acidic or basic waste streams utilizes a multiple compartmental basin usually constructed of concrete. This basin is lined with acid brick or coated with a material resistant to the expected environment.

In order to reduce the required volume of the neutralization basin, mixers are installed in each compartment to provide more intimate contact between the waste and neutralizing reagents, thus speeding up reaction time. Stainless steel plates mounted on the floor of the pit and directly below the mixers will reduce corrosion damage to the structure. Basin inlets are baffled to provide for flow distribution, while effluent baffles can help to prevent foam from being carried over into the receiving stream (Conway and Ross, 1980).

In some cases, neutralization may be accomplished in a discharge sewer.

10.1.8.2 Application/Limitation

Neutralization can be applied to any wastestream or wastewater requiring pH control. It is commonly used prior to biological treatment, since bacteria are sensitive to rapid pH changes and values outside a pH range of 6 to 9. Similarly, aquatic ecosystems are pH sensitive, therefore neutralization of wastewater is required prior to discharge to a receiving water body. In the case where hazardous wastes are hazardous because of corrosivity, neutralization may be required prior to acceptance for disposal. It is also used as a pretreatment for several chemical treatment technologies, including carbon adsorption, ion exchange, air stripping, wet air oxidation, and chemical oxidation/reduction processes. A pH adjustment is also dictated in several other situations, including protection of construction materials, breaking of emulsions, insolubilization of certain organic materials, and control of chemical reaction rates (e.g., chlorination) (Conway and Ross, 1980).

10.1.8.3 Design Considerations

The choice of an acidic reagent for neutralization of an alkaline wastewater is generally between sulfuric acid and hydrochloric acid. Sulfuric acid is usually used due to its lower cost. Hydrochloric acid has the advantage of soluble reaction end products.

The selection of a caustic reagent is usually between sodium hydroxide and various limes; ammonium hydroxide is occasionally used. The factors to be considered in choosing the most suitable reagent include: purchase cost, neutralization capacity, reaction rate, storage and feeding requirement, and neutralization products.

Although sodium hydroxide costs much more than the other materials, it is frequently used due to uniformity, ease of storage and feeding, rapid reaction rate, and soluble end products. The lime materials have the advantage of relatively low cost. This low material cost is at least partially offset by increased capital and operating costs for the rather complex feeding and reaction system required (Conway and Ross, 1980).

While the rate of reaction between the completely ionized sodium hydroxide and a strong acid waste is virtually instantaneous, the reactions of lime bases require considerable time for completion. Reaction time can be minimized by several approaches: a relatively high end point pH level, efficient mixing, and slurry feeding as opposed to dry feeding (Conway and Ross, 1980).

10.1.8.4 Technology Selection/Evaluation

Neutralization is a relatively simple unit treatment process which can be performed using readily available equipment. Only storage and reaction tanks with accessory agitators and delivery systems are required. Because of the corrosivity of the wastes and treatment reagents, appropriate materials of construction are needed to provide a reasonable service-life for equipment. The process is reliable provided pH monitoring units are used. The feed of the neutralization agent may be regulated automatically by the pH monitoring unit thereby ensuring effective neutralization and minizing worker contact with corrosive neutralizing agents.

Neutralization of hazardous wastes has the potential of producing air emissions. Acidification of streams containing certain salts, such as sulfide, will produce toxic gases. Feed tanks should be totally enclosed to prevent escape of acid fumes. Adequate mixing should be provided to disperse the heat of reaction if wastes being treated are concentrated. The process should be controlled from a remote location if possible.

10.1.8.5 Costs

Capital costs for a neutralization system include costs for chemical storage, chemical feeding and mixing. These costs can be approximated using Figure 10-7.

10.1.9 Gravity Separation

10.1.9.1 General Description

Gravity separation is a purely physical phenomenon in which the oil is permitted to separate from water in a conical tank.

10.1.9.2 Applications/Limitations

Gravity separators are primarily used to treat two-phased aqueous wastes. A typical application would be separation of free gasoline or fuel oil from a fuel contaminated aquifer. Gravity separation has also been used to separate PCB oils from contaminated groundwater. For efficient separation, the nonaqueous phase should have a significantly different specific gravity than water and should be present as a nonemulsified substance. Emulsion between water and oil is common, and an emulsion breaking chemical must frequently be added to the waste for efficient treatment.

10.1.9.3 Design Considerations

Gravity separators can take many shapes and arrangements, depending in part on the characteristics of the waste. The modern trend is to keep separator design small and simple to reduce costs. Typical design configurations include: horizontal cylindrical decanters, vertical cylindrical decanters, and cone-bottomed settlers.

Baffles are frequently installed to provide additional surface area, which promotes oil droplet coalescence. The cone-bottomed design is particularly useful if heavy solids are to be settled out of the waste while oil separation is proceeding.

10.1.9.4 Technology Selection/Evaluation

Gravity separation offers a straightforward, effective means of phase separation provided the oil and water phases separate adequately within the residence time of the tank. Simple, readily available equipment can be used and operational requirements are minimal. If emulsion-breaking chemicals must be added to promote oil-water separation, laboratory tests should be periodically conducted to ensure adequate dosing.

Consideration must also be given to the disposal of the extracted waste constituents collected. For gravity separation processes, this material consists of immiscible oil siphoned from the separator.

10.1.10 Air Stripping

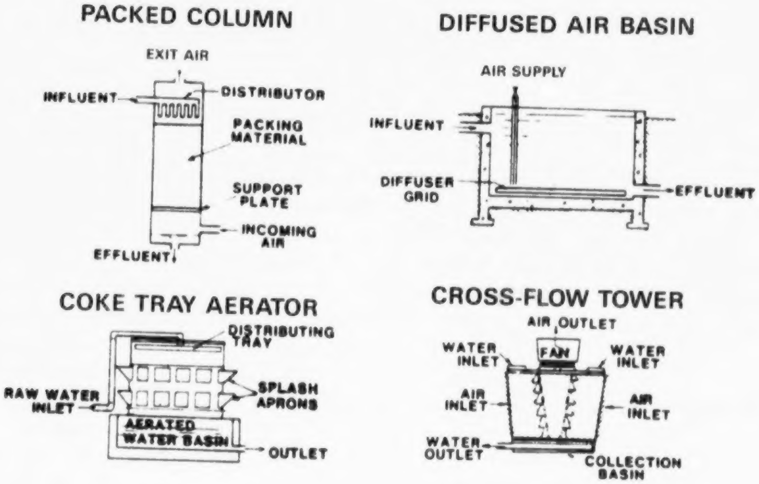
10.1.10.1 General Description

Air stripping is a mass transfer process in which volatile contaminants in water or soil are transferred to gas.

As shown in Figure 10-12, there are four basic equipment configurations used to air strip liquids.

Air stripping is frequently accomplished in a packed tower equipped with an air blower. The packed tower works on the principle of countercurrent flow. The water stream flows down through the packing while the air flows upward, and is exhausted through the top. Volatile, soluble components have an affinity for the gas phase and tend to leave the aqueous stream for the gas phase. In the cross-flow tower, water flows down through the packing as in the countercurrent packed column, however, the air is pulled across the water flow path by a fan. The coke tray aerator is a simple, low-maintenance process requiring no blower. The water being treated is allowed to trickle

FIGURE 10-12. AIR STRIPPING EQUIPMENT CONFIGURATIONS


PACKED COLUMN

DIFFUSED AIR BASIN

COKE TRAY AERATOR

CROSS-FLOW TOWER

Source: Canter and Knox, 1985

through several layers of trays. This produces a large surface area for gas transfer. Diffused aeration stripping and induced draft stripping use aeration basins similar to standard wastewater treatment aeration basins. Water flows through the basin from top to bottom or from one side to another with the air dispersed through diffusers at the bottom of the basin. The air-to-water ratio is significantly lower than in either the packed column or the cross-flow tower (Canter and Knox, 1985).

10.1.10.2 Applications/Limitations

Air stripping is used to remove volatile organics from aqueous wastestreams. Generally components with Henry's Law constants of greater than 0.003 can be effectively removed by air stripping (Conway and Ross, 1980). This includes such components as 1,1,1-trichloroethane, trichloroethylene, chlorobenzene, vinyl chloride, and dichloroethylene. The feed stream must be low in suspended solids and may require pH adjustment of hydrogen sulfide, phenol, ammonia, and other organic acids or bases to reduce solubility and improve transfer to the gas phase. Stripping is often only partially effective and must be followed by another process such as biological treatment or carbon adsorption. Combined use of air stripping and activated carbon can be an effective way of removing contaminants from groundwater. The air stripper removes the more volatile compounds not removed by activated carbon and reduces the organic load on the carbon, thus reducing the frequency (and expense) of carbon regeneration.

The countercurrent packed tower has been the most widely used equipment configuration for air stripping at hazardous waste sites. The reason for this are (Canter and Knox, 1985):

(1) It provides the most liquid interfacial area.

(2) High air-to-water volume ratios are possible due to low air pressure drop through the tower.

(3) Emission of stripped organics to the atmosphere may be environmentally unacceptable; however, a countercurrent tower is relatively small and can be readily connected to vapor recovery equipment.

The major disadvantage of the packed column is the high energy cost.

10.1.10.3 Design Considerations

The design of a packed tower air stripper generally involves a determination of the cross-sectional area of the column and the height of the column packing. The cross-sectional area of the column is determined from physical properties of the air flowing through the column, the characteristics of the packing and the air-to-water flow ratio.

A key factor is the establishment of an acceptable air velocity. A general rule of thumb used for establishing the air velocity is that an acceptable air velocity is 60% of the air velocity at flooding. Flooding is the condition in which the air velocity is so high that it holds up the water in the column to the point where the water becomes the continuous phase rather than the air. If the air-to-water ratio is held constant, the air velocity determines the flooding condition. For a selected air-to-water ratio, the cross-sectional area is determined by dividing the air flow rate by the air velocity. The selection of the design air-to-water ratio must be based upon experience or pilot-scale treatability studies. Treatability studies are particularly important for developing design information for contaminated ground water (Canter and Knox, 1985).

The height of column packing may be determined by the following equation (Canter and Knox, 1985):

$$Z = \frac{\ln\left[\frac{(X_2 - Y_1/H)}{(X_1 - Y_1/H)}(1-A) + A\right] L}{K_L aC(1-A)(1-X)M}$$

where Z = height of packing, ft.
L = water velocity, lb-mole/hr/ft^2
X_2 = influent concentration of pollutant in ground water, mole fraction
X_1 = effluent concentration of pollution in ground water, mole fraction
$K_L a$ = mass transfer coefficient, gal./hr.
C = molar density of water = 3.47 lb-mole/ft^3
H = Henry's law constant, mole fraction in air per mole fraction in water
G = air velocity, lb-mole/hr/ft^2
A = L/HG
(1-X)M = the average of one minus the equilibrium water concentration through the column
Y_1 = influent concentration of pollutant in air, mole fraction

In most cases, the following assumption can be made:

(1) $Y_1 = 0$, there should be no pollutants in the influent air.

(2) (1-X)M = 1, the influent concentrations should be too small when converted to mole fraction to shift this term significantly from 1.0.

The packing column height can then be determined by the simplified equation:

$$Z = \frac{\ln\left[\frac{X_2}{X_1} \quad (1-A) + A\right] L}{K_L a \; C(1-A)}$$

The mass transfer coefficient, $K_L a$, is determined from pilot-scale treatability studies, and is a function of type of compound being removed, air-to-water ratio, groundwater temperature, type of packing and tower geometry (Canter and Knox, 1985).

Calgon Carbon Corporation maintains a computer modelling system which determines the appropriate tower diameters, parking heights, air/water ratios and tower packing for a particular aplication (Calgon Carbon Corp. 1983). This system facilitates rapid mobilization of the packed tower equipment to a site.

10.1.10.4 Technology Selection/Evaluation

In recent years, air stripping has gained increasing use for the effective removal of volatile organics from aqueous wastestreams. It has been used most cost-effectively for treatment of low concentrations of volatiles or as a pretreatment step prior to activated carbon. Calgon manufacturers a treatment system which combines air stripping and activated carbon.

The equipment for air stripping is relatively simple, start-up and shut-down can be accomplished quickly, and the modular design of packed towers makes air stripping well suited for hazardous waste site applications.

An important factor in the consideration of whether to utilize air stripping technology for the removal of volatile contaminants is the air pollution implications of air stripping. The gas stream generated during treatment may require collection and subsequent treatment or incineration.

10.1.10.5 Costs

Packed tower air strippers have higher removal efficiencies than induced-draft systems, which are similar to diffused aeration systems. However, the induced-draft system is lower in capital cost and requires less energy to operate than a packed-tower system. Table 10-14 describes the installed cost of an induced-draft stripper manufactured and marketed by the Calgon Carbon Corporation. As shown in Table 10-14, the installed cost of an induced-draft stripper, capable of treating 700 gpm and removing 75 percent of the TCE contamination, is about 31 percent ($19,000 vs. $61,300) of the cost of a packed-tower capable of removing 95 percent of the TCE. Assuming that a well pump with a minimum discharge head of 25 pisig is required to feed both units, the packed-tower also uses an additional $5,100 per year in electrical energy for operation of the blower.

In a typical treatment system, re-pumping of the treated water would be required. Adding the cost of a sump, flow control, and a pump, the overall project cost for the induced-draft system would be about one-half the cost of the packed tower system (Calgon Carbon Corp., undated).

10.1.11 Oxidation

10.1.11.1 General Description

Reduction-oxidation (redox) reactions are those in which the oxidation-state of at least one reactant is raised while that of another is lowered. In

TABLE 10-14. AIR STRIPPING COST ESTIMATES
(Basis: 700 gpm; 1000 micrograms/liter TCE)

	Induced-Draft[1] Stripper (75% Removal)	Packed-Tower[2] 5-ft Diameter (95% Removal)
Air Stripping Equipment	$15,000	$42,300
Stripper Assembly & Installation	4,000	19,000
Equipment Sub-total	$19,000	$61,300
Recharge Pump; Assembly and Controls	$16,000	$16,000
Foundation/Sump[4]	18,000	23,700
Equipment Freight	2,000	5,000
Project Management	10,000	20,000
Project Contingency	7,000	20,000
Total	$72,000	$146,000

[1] Calgon Model No. 909B (8'0" x 9'1" x 9' 0").

[2] Tower is made of fiberglass reinforced plastic and contains 15 ft. of 2-in. diameter polypropylene pall ring packing.

[3] Cost includes tower, packing, packing support, demister, 4,000 cfm fan with 10 hp motor, damper, piping valves, and ductwork.

[4] Sump 5' x 5' x 8' below grade concrete.

Source: O'Brien and Stenzel, undated.

chemical oxidation, the oxidation state of the treated compound(s) is raised. For example in the conversion of cyanide to cyanate under alkaline conditions using permanganate, the oxidation state of the cyanide ion is raised as it combines with an atom of oxygen to form cyanate. This reaction can be expressed as follows:

$$2\ NaCN + 2KMnO_4 + KOH \quad 2\ K_2MNO_4 + NaCNO + H_2O$$

Common commercially available oxidants include potassium permanganate, hydrogen peroxide, calcium or sodium hypochlorite and chlorine gas.

10.1.11.2 Applications/Limitations

Chemical oxidation is used primarily for detoxification of cyanide and for treatment of dilute waste streams containing oxidizable organics. Among the organics for which oxidative treatment has been reported are: aldehyde, mercaptans, phenols, benzidine, unsaturated acids and certain pesticides (Kiang and Metry, 1982).

Chemical oxidation can be an effective way of pretreating wastes prior to biological treatment; compounds which are refractory to biological treatment can be partially oxidized making them more amenable to biological oxidation.

One of the major limitations with chemical oxidation is that the oxidation reactions frequently are not complete (reactions do not procede to CO_2 and H_2O). Incomplete oxidation may be due to oxidant concentration, pH, oxidation potential of the oxidant, or formation of a stable intermediate (Kiang and Metry, 1982). The danger of incomplete oxidation is that more toxic oxidation products could be formed. Chemical oxidation is not well suited to high-strength, complex waste streams. The most powerful oxidants are relatively non-selective and any oxidizable organics in the waste stream will be treated. For highly concentrated waste streams this will result in the need to add large concentrations of oxidizing agents in order to treat target compounds. Some oxidant such as potassium permanganate can be decomposed in the presence of high concentrations of alcohols and organic solvents (Kiang and Metry, 1982).

10.1.11.3 Design Considerations

Equipment requirements for chemical oxidation are simple and include contact vessels with agitators to provide suitable contact of the oxidant with the waste, storage vessels and chemical metering equipment. Some instrumentation is required to determine pH and the degree of completion of the oxidation reaction. Some oxidizing reagents react violently in the presence of significant quantities of readily oxidizable materials. Therefore reagents must be added in small quantities to avoid momentary excesses.

10.1.11.4 Technology Selection/Evaluation

Oxidation reactions can be carried out using simple, readily available equipment; only storage vessels, metering equipment, and contact vessels with agitators are required. However, implementation is complicated because every oxidation/reduction reaction system must be designed for the specific application. Laboratory- and/or pilot-scale testing are essential to determine the appropriate chemical feed rates and reactor retention times in accordance with reaction kinetics. Oxidation and reduction has not been widely used in treating hazardous wastestreams.

A major consideration in electing to utilize oxidation technology is that the treatment chemicals are invariably hazardous, and great care must be taken in their handling. In particular, the handling of many oxidizing agents is potentially hazardous and suppliers' instructions should be carefully followed.

In some cases, undesirable byproducts may be formed as a result of oxidation. For example, addition of chlorine can result in formation of bio-resistant end products which can be odorous and more toxic than the original compound. The possibility of this undesirable side reaction needs to be considered when using chlorine for oxidation of wastewaters (Conway and Ross, 1980).

10.1.11.5 Costs

Capital costs for chemical oxidation include costs for chemical storage, chemical feeding and chemical mixing. These costs can be approximated using Figure 10-7. Chemical costs are listed in Table 9-10.

10.1.12 Chemical Reduction

10.1.12.1 General Description

Chemical reduction involves addition of a reducing agent which lowers the oxidation of a substance in order to reduce toxicity or solubility or to transform it to a form which can be more easily handled. For example, in the reduction of hexavalent chromium (Cr(Vl)) to trivalent chromium (Cr(III)) using sulfur dioxide the oxidation state of Cr changes from 6+ to 3+ (Cr is reduced) and the oxidization state of S increases from 2+ to 3+ (S is oxidized). The decrease in the positive valence or increase in the negative valence with reduction takes place simultaneously with oxidation in chemically equivalent ratios (Kiang and Metry, 1982).

$$2H_2CrO_4 + 3SO_2 + 3H_2O \quad Cr_2(SO_4)3 + 5\ H_2O$$

Commonly used reducing agents include sulfite salts (e.g. sodium bisulfite, sodium metabisulfite, sodium hydrosulfite) sulfur dioxide and the base metals (iron, aluminum and zinc).

10.1.12.2 Applications/Limitations

Chemical reduction is used primarily for reduction of hexavalent chromium, mercury and lead. There are currently no practical applications involving reduction of organic compounds.

10.1.12.3 Design Considerations

Very simple equipment is required for chemical reduction. This includes storage vessels for the reducing agents and perhaps for the wastes, meterring equipment for both streams, and contact vessels with agitators to provide suitable contact of reducing agent and waste. Some instrumentation is required to determine the concentration and pH of the waste and the degree of completion of the reduction reaction. The reduction process may be monitored by an oxidation-reduction potential (ORP) electrode (Kiang and Metry, 1982).

10.1.12.4 Technology Selection/Evaluation

Chemical reduction is well demonstrated for the treatment of lead, mercury and chromium. However, for complex waste streams containing other potentially reducible compounds, laboratory and pilot scale tests will be required to determine appropriate chemical feed rates and reactor retention times.

Chemical reduction can be carried out using simple, readily available equipment and reagents. Capital and operating costs are low and the process is easy to implement.

10.1.12.4 Costs

Capital costs for chemical reduction include costs for chemical storage, chemical feeding, and chemical mixing. These costs can be approximately using Figure 10-7.

10.2 Solids Treatment

10.2.1 Solids Separation

This section describes equipment and methods used to separate solids from slurries, and/or to classify contaminated soils or slurries according to grain size. The objective of separating solids from slurries is to attain two distinct waste streams: a liquid waste stream that can be subsequently treated for removal of dissolved and fine suspended contaminants; and a concentrated slurry of solids and minimal liquid that can be dewatered and treated.

Classification of particles according to grain size may be undertaken for one of two reasons. The first reason is that more efficient use can be made of equipment and land area by taking advantage of the differences in settling velocity of different sized particles. For example, where only limited land space is available, settling basins may be used to remove sand and gravel with a high settling velocity and then high rate gravity settlers could be used to remove fine-grained particles.

There is recent evidence to suggest that classification by grain size is important in managing hazardous waste contaminated soils and sediments because of the apparent tendency of contaminants to adsorb preferentially onto fine-grained materials such as clay and organic matter. The separation of solids by grain size and level of contamination could prove to be extremely beneficial to the overall management (treatment, transport, and disposal) of contaminated soil material. Whereas relatively non-contaminated soils and sediments may be disposed of in ordinary sanitary landfills or discharged back into the stream, the highly contaminated solids must be disposed in a hazardous waste landfill, incinerated or treated to render them non-hazardous.

The most appropriate solids separation method for a given site depends upon several factors, including the following:

- Volume of contaminanted solids
- Composition of soils or sediments, including gradation, percent clays, and percent total solids
- Types of dredging or excavation equipment used, which determines the feed rate to solids separation and, in the case of slurries, the percent solids
- Site location and surroundings. The available land area and ultimate or present land use may limit the type of system that can be utilized.

Solids separation methods addressed in this section include: sieves and screens, hydraulic and spiral classifiers, cyclones, settling basins and clarifiers.

Many of the methods presented in this section are discussed in terms of their ability to treat soils slurries or sediments containing particle within a specific size range. Table 10-15 summarizes the particle sizes which correspond to various soil types.

10.2.1.1 Screens and Sieves

Sieves or screens consist of bars, woven wire or perforated plate surfaces which retain particles of a desired size range while allowing smaller particles and the carrying liquid to pass through the openings in the screening surface. Several types of screens and sieves have application for solids separation at hazardous waste sites.

a. Grizzlies

General Description--Grizzlies consist of parallel bars which are frame-mounted on an angle to promote materials flow and separation. Hoppers are provided beneath the grizzly to collect removed material. Bar spacing is generally 1 to 5 inches apart depending upon the desired separation. Both

TABLE 10-15. APPROPRIATE PARTICLE SIZES FOR VARIOUS SOIL CATEGORIES

USCS Classification	U.S. Standard Sieve Size
Gravel	
Coarse	>3/4 in.
Fine	No. 4 - 3/4 in.
Sand	
Coarse	No. 10 - No. 4
Medium	No. 40 - No. 10
Fine	No. 200 - No. 40

USCS Classification	Particle Size (u)
Silt	10 - 74
Coarse Clay	1.0 - 10
Fine Clay	<1.0

fixed and vibrating grizzlies are available. Grizzlies generally have a maximum width of 6 to 9 feet and a length of 12 to 18 feet (Mallory and Nawrocki, 1974).

Applications/Limitations--Grizzlies are used primarily for scalping, i.e., removing a small amount of oversized material from a stream which is predominantly fines. They are generally limited to separating materials which are 2 inches in diameter or coarser. Another major function of the grizzly is to reduce velocity of a slurry for subsequent processing operations (Mallory and Nawrocki, 1974).

Technology Selection/Evaluation--Grizzlies offer a reliable method for removing coarse-grained material from slurries. By doing so, they significantly improve the reliability and performance of subsequent solids separation methods and also reduce maintenance costs by minimizing the amount of abrasive material which reaches the screen, cyclone, etc. Grizzlies contain no moving parts and are tough and abrasion resistant. Therefore maintenance requirements are minimal. Space requirements are also minimal and they can be installed in almost any area. They can easily be arranged in series or parallel to accommodate very high flows or achieve classification of coarse materials.

a. Moving Screens

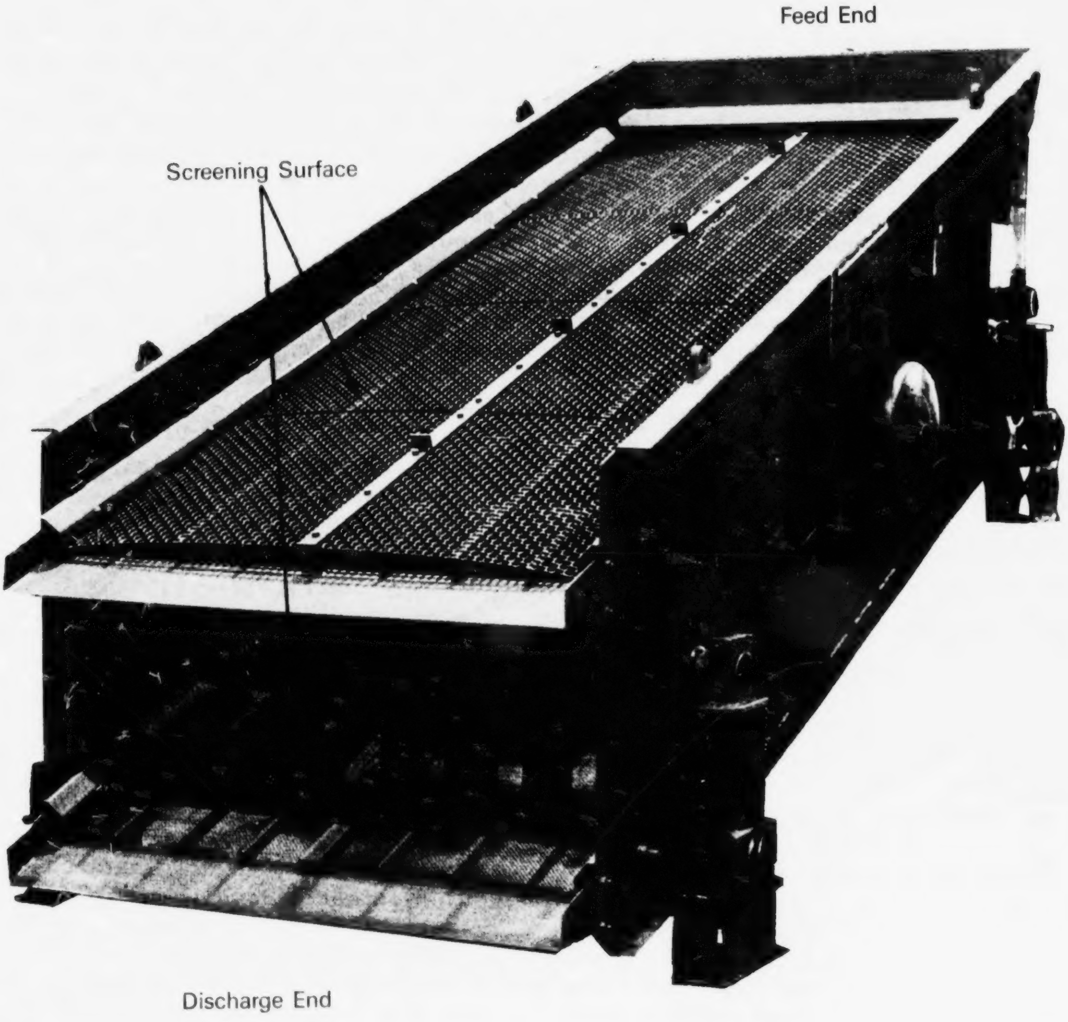
General Description--Screening of fine particles from dry materials is frequently accomplished using moving screens. Types of moving screens include:

- vibrating screens
- revolving screens
- gyratory screens

Vibrating screens are more widely used than other screen types, particularly for fine particle separation, because of their larger capacity per unit of screen area and their higher efficiency (Perry and Chilton, 1973). Only the vibrating screen will be described in this section.

Vibrating screens consist of a plane screening surface, usually stretched tautly and set into a rectangular frame having sufficient sidewalls to confine the material flow. Figure 10-13 illustrates a typical vibrating screen. They may be composed of one, two or three screening decks. This allows for progressively finer separation and lower space requirements. Screens are usually inclined at a slope of approximately 20° from horizontal, although horizontal screens are also available. Vibration is produced by circular motion in a vertical plane. By vibration, the bed of material tends to develop a fluid state. Larger particles remain on top of the bed while smaller particles sift through the voids and find their way to the bottom. Once the fine particles have sifted through the bed of material, the vibrating action increases the probability that the small particles will pass through

FIGURE 10-13. TYPICAL VIBRATING SCREEN



Source: Allis-Chalmers Corp., undated.

the screen. An inclined screen allows the material to cascade down the screen surface, increasing the probability that small particles will pass through (Allis Chalmers, undated).

Vibrating screens typically range in size from about 3 to 10 feet wide and 6 to 30 feet long. Solids handling capacity ranges from 300 to 950 tph.

Applications/Limitations--The function of vibrating screens is to separate particles by grain size. The oversized particles are substantially dewatered during the separation. Typically, vibrating screens are used to separate materials in the size range of 1/8 inch up to 6 inches. However, high speed vibrating screens are also available for separating finer particles in the size range of 4 to 325 mesh (Chilton and Perry, 1973; Mallory and Nawrocki, 1974). Although separation efficiencies are high with the vibrating screen, some fine particles are invariably carried over with the coarse particles. Conventional vibrating screens are best suited for handling dry materials. Wet or sticky materials tend to blind the screen. Larger openings can be used where blinding is a problem, but this reduces the efficiency of the size separation. Vibrating screens with heated decks are also available to reduce moisture content, although they are not cost-effective for waste streams with a high moisture content. Because of these limitations, the conventional vibrating screens are not well suited for handling dredge slurries. Where the moisture content of the material is high resulting in blinding wet screening with sprays can be used. Water is generally sprayed at 3 to 6 gpm per ton at a minimum of 20 psi to discourge blinding (Allis Chalmers, undated).

The presence of abrasive material in the feed may result in the need for frequent screen replacement. Therefore, wastes should be carefully prescreened using a grizzly or wedge-bar screen.

Relative to other types of moving screens, vibrating screens generally are the most efficient, have lowest space requirements and lowest maintenance costs. Vibrating screens are the most efficient of the moving screens for separating solids according to grain size. However, their reliability is adversely affected by the fact that wet or sticky materials tend to blind the screen. A water spray applied to a vibrating screen can significantly reduce blinding. The effectiveness of vibrating screens should be determined on a case-by-case basis.

The presence of abrasive material can result in the need for frequent screen replacement thereby increasing maintenance costs.

Vibrating screens are relatively compact. They can be installed in areas where space is limited and are well suited for use in mobile treatment systems.

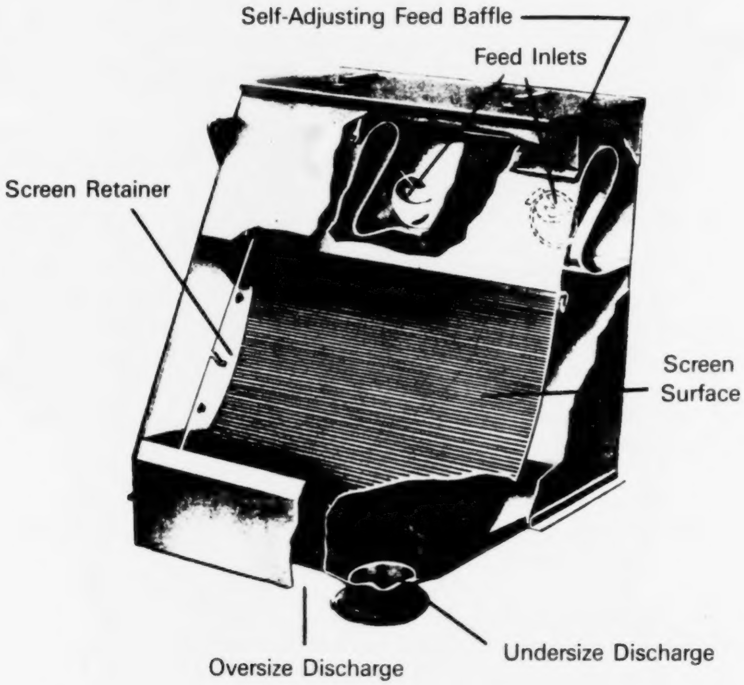
Cost--Costs for vibrating screens vary with the size and capacity of the screens, The capital cost for a 10-ft. long, 5-ft. wide, 5-ft. high screen with a capacity of 200 TPH is about $25,000. Operation and maintenance costs for vibrating screens are relatively low compared to other types of moving screens (Allis Chalmers, undated; Derrick Manufacturing Corp., undated).

c. Stationary or fixed screens

Stationary or fixed screens differ from moving screens in that they posess no moving parts. A continuous curved surface and the velocity of the slurry across the surface provide a centrifugal force which holds the slurry against the screens and allows for separation. One type of fixed screen which has potential application for solids separation at hazardous waste sites is the wedge-bar screen or bend-sieve. A typical wedge-bar screen is illustrated in Figure 10-14. The hydrosieve, a modified wedge-bar screen which uses water pressure to encourage solids separation is also used.

General Description--The wedge-bar screen is similar in design to a grizzly insofar as it consists of parallel bars which are frame-mounted on a curved deck. However, in the case of the wedged bar screen, bar spacing is very close to effect fine particle separation. As the material enters the feed inlet, a series of baffles in the feed box spread the material so that the slurry is evenly fed over the width of the curved screen deck. The slurry

FIGURE 10-14. WEDGE BAR SCREEN



Source: Dorr-Oliver, 1983

flows through the feed inlet at the top of the feed box and flows tangentially down the surface of the screen. The continuous curved surface together with the velocity across the surface provides a centrifugal force which holds the slurry against the screen surface. As the slurry strikes the sharp edge of the wedge bar, small particles are sliced off and directed downward through the slots along with most of the liquid. Dewatered, oversized material slides on top of the screen surface and is discharged. The slicing action of the wedge bars sizes the undersize particles at a smaller dimension than the slots themselves and helps to minimize blinding. For example, for a slot width of 1 mm, the thickness of the slurry layer being shaved off is about 1/4 mm. This 1/4 mm thick cut can transport particles of up to 1/2 mm in size; plus 1/2 mm solids pass over the screen (Hoffman-Muntner Corporation., 1978; Dorr-Oliver, 1980; Dorr-Oliver, 1983).

Wedge-bar screens normally come in sizes of 2 to 6 feet wide, with capacities of 30 to 200 gpm/ft^2.

The hydrosieve or pressure screen is a modification of the conventional wedge-bar screen in which the pressure of a water spray encourages more efficient separation. The water pressure helps to remove fines that are adhering to coarse grain sized materials and breaks up clumps of material which tend to clog the screen. Hydrosieves with capacities of up to 1500 gpm are available.

Application/Limitation--Wedge-bar screens and hydrosieves are used to separate particles in slurry by grain size. The wedge-bar screen is generally less efficient in separating solids than the vibrating screen; the oversized material typically carries a considerable amount of fines. The hydrosieve minimizes this problem by employing a pressure spray which washes the fines from the coarser material. Wedge bar screens may be used ahead of vibrating screens. This provides a higher solids separation efficiency than the vibrating screen alone (Allis Chalmers, undated).

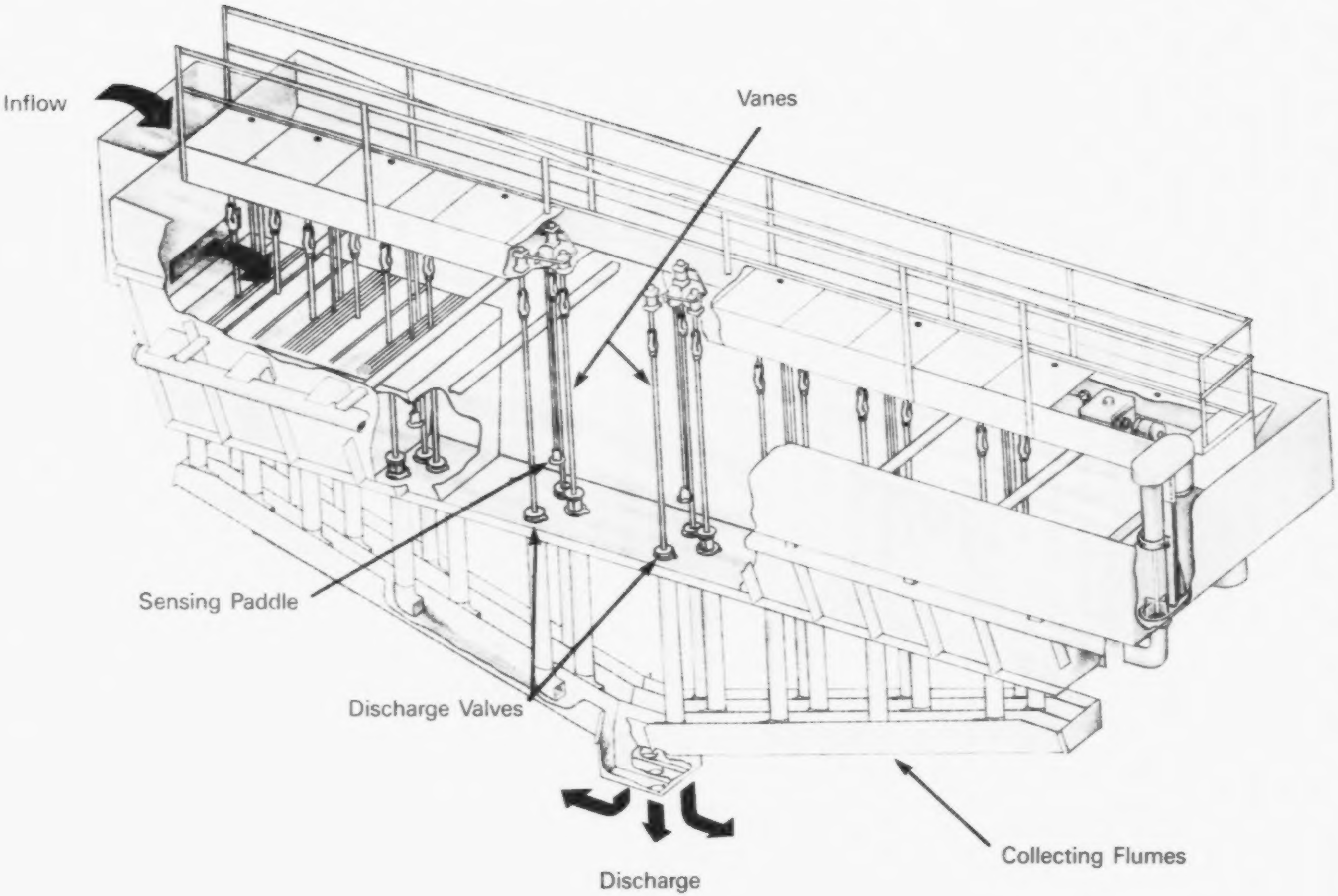
Technology Selection/Evaluation--The wedge-bar screen offers a very low cost method for separating solids according to grain size. However, the effectiveness of the separation methods is not as good as that achieved using vibrating screens or cyclones. Nevertheless, use of a water spray with a wedge-bar screen (hydrosieve) can significantly improve separation efficiency by removing fines which are sorbed to sands and gravel. The wedge-bar screen contains no moving parts and is extremely easy to operate and maintain. It is also more resistant to abrasion than the vibrating screen. It is compact and requires a minimal amount of space.

10.2.1.2 Hydraulic Classifiers

a. General Description

Hydraulic classifiers are commonly used to separate sand and gravel from slurries and classify them according to grain size. A typical hydraulic classifier is shown in Figure 10-15. These units consist of elevated

FIGURE 10-15. HYDRAULIC CLASSIFIER



Source: Eagle Iron Works, 1982

rectangular tanks with v-shaped bottoms to collect the material. Discharge valves which are located along the bottom of the tank are activated by motor-driven vanes that sense the level of solids as they accumulate. The principal of operation is simple. The slurry is introduced into the feed end of the tank. As the slurry flows to the opposite end, solids settle out according to particle size as a result of differences in settling velocity. Coarse materials settle out first near the feed end and materials are progressively finer along the length of the tank. Manually adjusted splitter gates below the discharge valves can be used to selectively direct materials of specific grain sizes to subsequent handling and treatment (Eagle, 1981; Mallory and Nawrocki, 1974). Classifying tanks are generally available in sizes ranging from 8 to 12 feet wide and 20 to 48 feet long (Mallory and Nawrocki, 1974). Solids handling capabilities are generally limited to 250 to 350 tph (Mallory and Nawrocki, 1974; Eagle Iron Works, 1981).

b. Applications/Limitations

Hydraulic classifiers are used to remove sand and gravel size particles from slurries and to classify the removed materials according to grain size. Materials are recovered from the classifier at about 30 percent moisture content (Written communication, 1984 Eagle Iron Works, Des Moines, Iowa). They are capable of removing and classifying materials within the size range of 3/8 inch down to about 150 to 200 mesh (105 to 74 microns) (Mallory and Nawrocki, 1974; Eagle Iron Works, 1981). The upper limitation of 3/8 inch is handled by prescreen- ing the wastes to remove all large materials. Other solids separation techniques are required to classify the fine-grained materials (<200 mesh). Another limitation is that some fines will be removed with the sand and gravel fraction. This limitation is frequently overcome by directing the solids to a spiral classifier where they are washed to remove the fine-grained materials (see Section 10.2.1.3). Hydraulic classifers have a relatively low solids handling capacity and are not well suited for handling large volumes of flow or high-solids concentrations. A single average sized tank with dimensions of 36 feet by 10 feet, for example, can handle 5300 gpm when separating material down to 100 mesh and only 1400 gpm when separating material down to 200 mesh (Eagle Iron Works, 1981).

Because of the inability of hydraulic classifiers to handle large volumes of flow, a combination of solids separation methods may be advisable to reduce the number of hydraulic classifiers needed for a large solids handling operation. One possibility for reducing the number of classifers needed would be to use these units to separate only those particles larger than 105 microns. Cyclones, hydrocyclones, or hydrosieves (see Sections 10.2.1.4 and 10.2.1.1) could then be used to remove the fine sand fraction (Mallory and Nawrocki, 1974).

c. Technology Selection/Evaluation

Hydraulic classifers offer an effective method for operating and classifying particles ranging in size from fine gravel to fine sands. Some fines are inadvertently removed with the sand and gravel, and the effectiveness of the separation can be improved by washing the collected solids in a spiral classifier to remove the fines.

Hydraulic classifier tanks are generally designed and sized to be truck mounted for mobile system applications. Start-up and shut-down can be accomplished quickly. Maintenance requirements are fairly simple.

Use of hydraulic classifiers can be easily integrated with other solids separation methods and this is advisable where large flows are involved or where classification of fine-grained materials (clays, silts) is required.

d. Costs

Costs for hydraulic classifiers vary with size and capacity of the classifier. For a size range of 24 to 49 feet long, 8 to 12 feet wide, and 8 feet deep; and a feed rate of 200 to 350 TPH, the initial cost ranges from $30,000 to $76,000 (Eagle Iron Works, 1981; Mallory and Nawroki, 1974).

10.2.1.3 Spiral Classifier

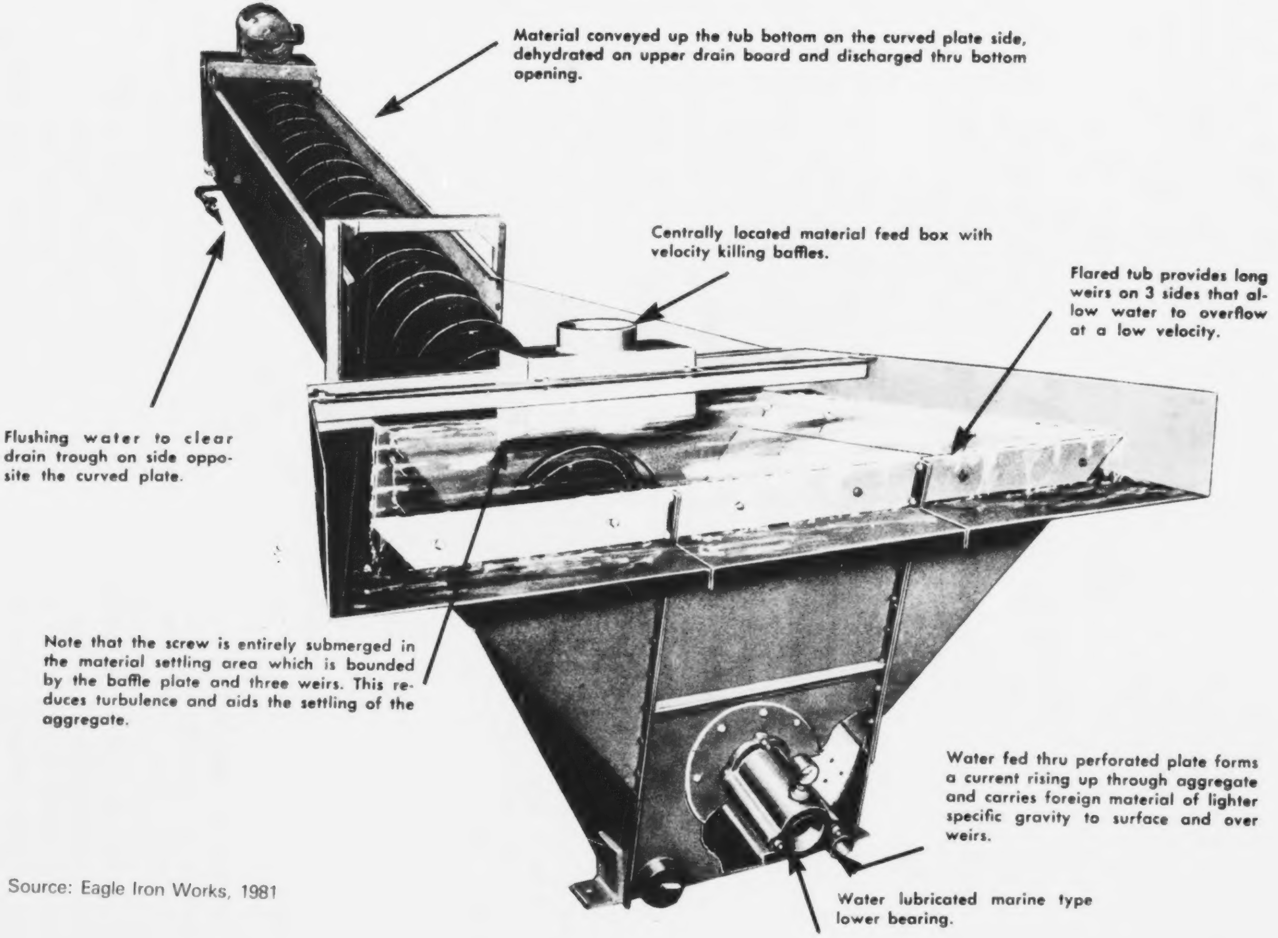
a. General Description

The spiral classifier consists of one or two long, rotating screws, mounted on an incline within a rectangularly shaped tub. It is used primarily to wash adhering clay and silt from sand and gravel fractions. Figure 10-16 shows a typical configuration of a spiral classifier.

The screw conveys settled solids from a hydraulic classifier (Section 10.2.1.2) up an incline to be discharged through an opening at the top of the tub. Fines and materials of low specific gravity are separated from sand and gravel through agitation and the abrading and washing action of the screw, and are removed along with the wastewater overflow at the bottom of the tub. The tumbling and rolling action caused by the continuous screw grinds particles against each other and removes the deleterious material coating the sand particles. This tumbling action also aids in dewatering materials by breaking the moisture film on the sand particles. As the moisture is relieved of surface tension, it is free to drain from the material (Eagle Iron Works, 1982). The sands which are finally discharged are substantially dewatered.

In general, the greater the length of the tub the higher the degree of dewatering and the greater the screw diameter the larger the capacity of the spiral classifier (Eagle Iron Works, 1982). Classifiers are available which are capable of handling up to 950 tph.

FIGURE 10-16. SPIRAL CLASSIFIER


Material conveyed up the tub bottom on the curved plate side, dehydrated on upper drain board and discharged thru bottom opening.
Centrally located material feed box with velocity killing baffles.
Flared tub provides long weirs on 3 sides that allow water to overflow at a low velocity.
Flushing water to clear drain trough on side opposite the curved plate.
Note that the screw is entirely submerged in the material settling area which is bounded by the baffle plate and three weirs. This reduces turbulence and aids the settling of the aggregate.
Water fed thru perforated plate forms a current rising up through aggregate and carries foreign material of lighter specific gravity to surface and over weirs.
Water lubricated marine type lower bearing.

Source: Eagle Iron Works, 1981

b. Applications/Limitations

Spiral classifiers are used primarily to wash, dewater, and classify sands and gravels up to 3/8 inch in diameter. They are not a singularly viable solids separation technology, but they are effective when used together with the hydraulic classifier. Spiral classifiers have a large capacity and are completely portable.

c. Technology Selection/Evaluation

Spiral classifiers improve the efficiency of solids separation achieved with the hydraulic classifier by removing fine grained materials attached to coarser particles.

Spiral classifers are generally designed to be trailer mounted for use in mobile treatment systems. Start-up and shut-down can be accomplished quickly and maintenance requirements are simple.

d. Costs

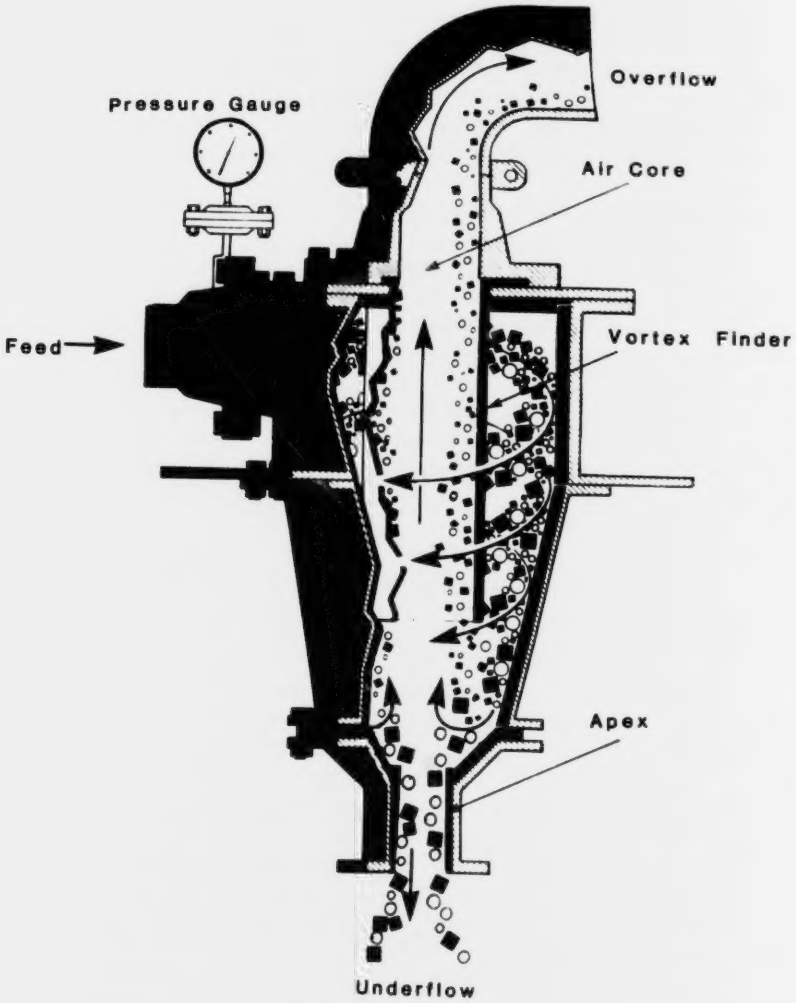
Costs for spiral classifiers vary with size and configuration. For a size range of 22 to 34 feet long, 8 to 19 feet wide, and 8 to 12 feet high the initial cost of a spiral classifier ranges from $14,000 to $77,000 for a single-screw-type; and from $37,000 to $150,000 for a double-screw-type. Operational and maintenance costs vary with the type of power utilized; it can be electricity, gas, or diesel fuel (Eagle Iron Works, 1982; Mallory and Nawroki, 1974).

10.2.1.4 Cyclones and Hydrocyclone

a. General Description

Cyclones and hydrocyclones are separators in which solids that are heavier than water are separated by centrifugal force. The major components of a hydrocyclone are shown in Figure 10-17. A hydrocyclone consists of a cylindrical/conical shell with a tangential inlet for feed, an outlet for the overflow of slurry, and an outlet for the underflow of concentrated solids. Cyclones and hydrocyclones contain no moving parts. The slurry is fed to the unit with sufficient velocity to create a "vortex" action that forces the slurry into a spiral and, as the rapidly rotating liquid spins about the axis of the cone, it is forced to spiral inward and then out through a centrally located overflow outlet. Smaller-sized particles remain suspended in the liquid and are discharged through the overflow. Larger and heavier particles of solids are forced outward against the wall of the cone by centrifugal force

FIGURE 10-17. TYPICAL CYCLONE



Source: Krebs Engineers, undated

within the vortex. The solids spiral around the wall of the cyclone and exit through the apex at the bottom of the cone (Dorr-Oliver, 1984).

Cyclones are available in an extremely wide range of sizes. The smallest units handle flows of only a few gallons per minute, while the largest units can handle between 2000 and 7000 gpm, depending upon slurry composition (Dorr-Oliver, 1984; Krebs Engineers, undated). However, cyclones do not scale-up as many other equipment types do. In general, the larger the cyclone diameter and inlet, the coarser the separation and the greater the cyclone capacity. The smaller the diameter and inlet, the finer the separation and the lower the hydraulic capacity. In order to remove small particles from large volume slurries, it is necessary to use multiple, small-diameter cyclones connected in parallel. Banks of multiple cyclones, manufactured as a single unit with a single feed pipe, are commercially available.

Cyclones can also be connected in series or in various staging arrangements to accomplish different objectives. For example, a high degree of particle size separation can be achieved by employing a bank of cyclones in series with decreasing cyclone size and particle size removal in the direction of flow. It is also possible to achieve a higher underflow concentration and a more clarified overflow by staging the cyclones. The first stage of cyclones could be used to classify the solids according to the desiged grain size. The second stage overflow cyclone could serve as a clarifier and the underflow cyclone could serve as the concentrator. However, the maximum underflow concentration achievable with cyclones is about 60 percent, since some liquid is necessary for solids discharge (Dorr-Oliver, undated).

It should be noted that cyclones are available which can handle some variation in flow rate and particle size by interchanging certain parts of the cyclone. For example, it is possible to add or delete sections to the cone, or to change the size of the vortex finder.

b. Applications/Limitations

Cyclones are available for separating or classifying solids over a broad particle size range, from 2000 microns down to 10 microns. However, in hazardous waste site applications they would be used primarily to remove smaller size particles from slurries and in situations where a sharp separation by particle size is needed. They are particularly applicable to situations where space is limited.

Cyclones are generally not effective for slurries with a solids concentration greater than 30 percent, for highly viscous slurries, or for separation of particle sizes with a specific gravity of less than about 2.5 to 3.2 (Krebs Engineers, undated). Slurries with a high clay content exhibit high pseudoplasticity or high viscosity and cannot be effectively removed using cyclones or hydrocyclones (Oklahoma State University, 1973).

Cyclones are highly vulnerable to clogging by oversized particles, and a high degree of prescreening (or use of progressively smaller cyclones in series) may be needed to avert clogging.

c. Technology Selection/Evaluation

Cyclones offer an effective means of separating and classifying solids over a broad range of particle size, provided the solids concentration is not too high and the slurry is not too viscous. Cyclones are flexible insofar as they can easily be arranged in parallel to accomplish fine size separation, or in various series or staging arrangements to improve classification of the overflow or concentration of the underflow. They can also be easily integrated with other solids separation methods. However, each individual cyclone is capable of handling only very limited variations in flow rate and particle size.

The capital and operating costs of cyclones are relatively low. They are simple to operate and easy to maintain since they contain no moving parts. Liners require periodic replacement but this can be done easily.

Cyclone assemblies take up less space than most solids separation equipment and are well-suited for tight locations. Because of their compactness and simplicity of operation, cyclones are also well-suited for inclusion in mobile treatment systems.

d. Costs

The cost of cyclones varies widely according to the size and the number of cyclones placed in series. The feed rate can vary from a few gallons per minutes up to several thousand gallons per minute, and the size of each cyclone can vary from 1/2 inch to 30 inches in diameter. Initial costs for cyclones can be as low as $5,000 and indefinately high, depending on the configuration (Hoffman Muntnor Corp., 1978; Krebs Engineers, undated).

10.2.1.5 Settling Basin

A settling basin, as described in this section, is an impoundment, basin, clarifier, or other container that provides conditions conducive to allowing suspended particles to settle from a liquid by gravity or sedimentation. The slurry is introduced into the basin and settling of solids occurs as the slurry slowly flows across the length of the basin. Flow out of the opposite end of the basin is reduced in its solids content.

The size of an impoundment basin or clarifier is ideally determined by dividing the critical settling velocity by the overflow rate. The critical settling velocity is a function of the diameter, and specific gravity of the smallest particle size requiring removal and the viscosity of the water.

However, ideal settling conditions are never achieved and the actual design of the required surface area must make allowance for turbulence, short circuiting, and scour velocity. Detailed procedures for sizing sedimentation basins can be found in most wastewater engineering handbooks.

Settled solids accumulate on the bottom of settling basins where they are temporarily stored. As the volume of accumulated solids increases, the effective size of the basin decreases, reducing the basin's effectiveness or efficiency. Accumulated solids must be periodically or continuously removed in order for the basin to perform as intended.

Commonly used types of settling basins are described below:

a. Impoundment Basin

General Description -- An impoundment basin is an earthen impoundment or diked area that is lined in a manner that is appropriate for protecting underlying groundwater. An adjustable weir is provided to control overflow rate. A typical impoundment basin is illustrated in Figure 10-18.

Multiple basins, or bulkheads that separate a single basin into compartments can be used in parallel to allow continuous sediment/water separation while accumulated solids are being removed from individual basins. Multiple basins can also be connected in series in order to separate solids according to grain size. Each basin would be designed to retain sediments of increasingly smaller grain size.

Applications/Limitations -- Impoundment basins are used to remove particles in the size range of gravel down to fine silt (10 to 20 microns with flocculants) (Mallory and Nawrocki, 1974). They are also used to provide temporary storage of dredged material and to classify sediment particles according to grain size.

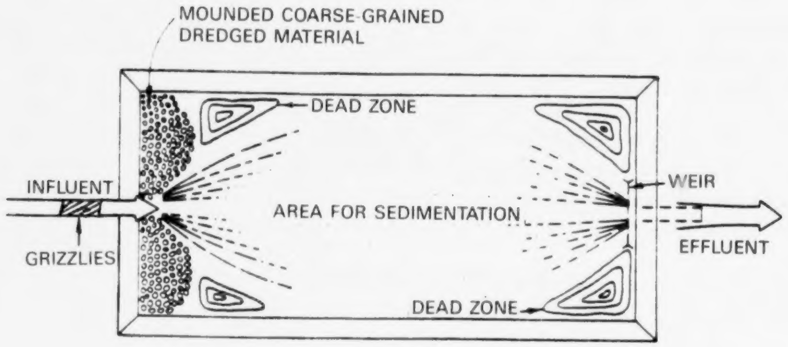
Impoundment basins are particularly well-suited for large-scale dredging operations, provided there is adequate land space available for their construction. They are not suitable for congested areas, or for areas where adequate measures cannot be taken to protect groundwater supplies (e.g., high groundwater table).

A major limitation with the use of impoundment basins is that unlike clarifiers, they have no mechanism for solids collection. Therefore, mechanical dredges (e.g., clamshells, backhoes) are typically used to remove the settled solids. This greatly increases the operational costs associated with use of impoundment.

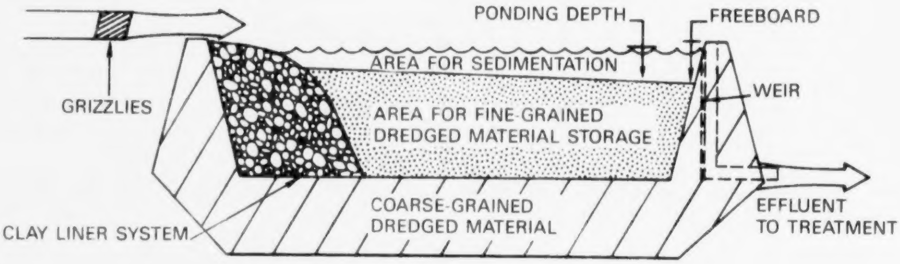
b. Conventional Clarifers

General Description -- Conventional clarifers are rectangular or circular settling basins which are typically equipped with built-in solids collection and removal mechanisms.

FIGURE 10-18.
CONCEPTUAL DIAGRAM OF A DREDGED MATERIAL CONTAINMENT AREA



PLAN



CROSS SECTION

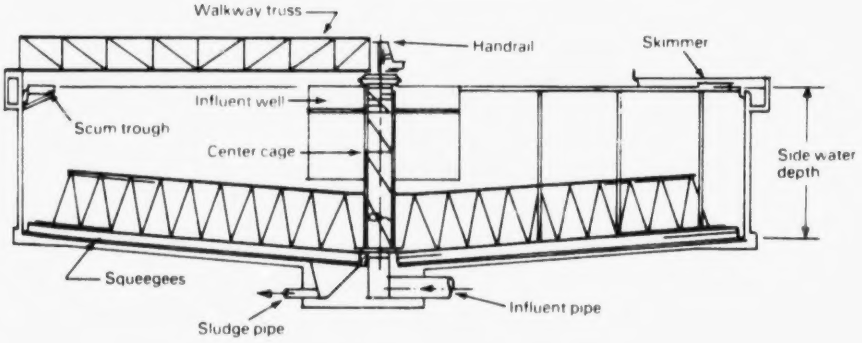
Source: Adapted from Palermo et al., 1978

Typically, in a rectangular clarifier a flow with relatively high suspended solids is introduced at one end of the clarifier, solids settle along the length of flow, and a flow with relatively low suspended solids leaves the clarifiers through trough-type overflow weirs. In most rectangular clarifiers flights extending the width of the tank move the settled sludge toward the inlet end of the tank. Some designs move the sludge toward the effluent end of the tank, corresponding to the direction of flow of the density current.

Circular clarifiers are of two general types. With the center feed type, the waste is fed into a center well and the effluent is pulled off at the weir along the outside. With a peripheral feed tank, the effluent is pulled off at the tank center.

Figure 10-19 illustrates a center feed type clarifier. The circular clarifer can be designed for center sludge withdrawal or vacuum withdrawal over the entire tank bottom.

FIGURE 10-19. CIRCULAR CLARIFIER



Source: Dorr-Oliver, 1976

Many clarifiers are equipped with separate zones for chemical mixing and precipitation, flocculation and settling.

Applications/Limitations -- Clarifiers are able to remove particles down to 10 to 20 microns (Mallory and Nawrocki, 1974) in diameter,with the use of flocculants. They can also be used to produce a thickened sludge with a solids concentration of about 4 to 12 percent (Metcalf and Eddy, 1979) and to separate solids by grain size. This would be accomplished by connecting clarifiers in series and providing a retention time sufficient to removal materials of a certain grain size.

Clarifiers are best suited to small- to moderate-scale cleanup operations or to large-scale operations where impoundment basins will not adequately protect groundwater supplies. Clarifiers can be barge mounted for solids separation during dredging operations.

Circular clarifiers are generally more efficient in solids removal. However, rectangular tanks are more suitable for barge-mounting and where construction space is limited. In addition a series of rectangular tanks is cheaper to construct due to the shared wall concept.

c. High Rate Clarifier

General Description -- High rate clarifiers use multiple "stacked" plates, tubes, or trays to increase the effective settling surface area of the clarifier and decrease the actual surface area needed to effect settling. Figure 10-20 illustrates a high rate clarifier. High rate clarifiers allow a higher flow rate per unit of actual surface area (loading rate) than do conventional clarifiers, thus the name "high rate" clarifiers. The trays, plates, or tubes also induce optimum hydraulic characteristics for sedimentation by guiding the flow, reducing short circuiting and promoting better velocity distribution.

High rate clarifiers are able to handle between 2 to 10 times the loading rate of conventional clarifiers and therefore require limited land use (Jones, Williams and Moore, 1978). Package units capable of handling 1,000 to 2,000 gpm are available and are easily transportable by truck or barge.

Applications/Limitations -- High rate clarifiers are best suited to small- to moderate-scale cleanup operations, or to large-scale operations where construction of earthen impoundments will not adequately protect groundwater. High rate clarifiers are particularly applicable to cleanup operations where land space is limited and where barge mounting of clarifiers is required.

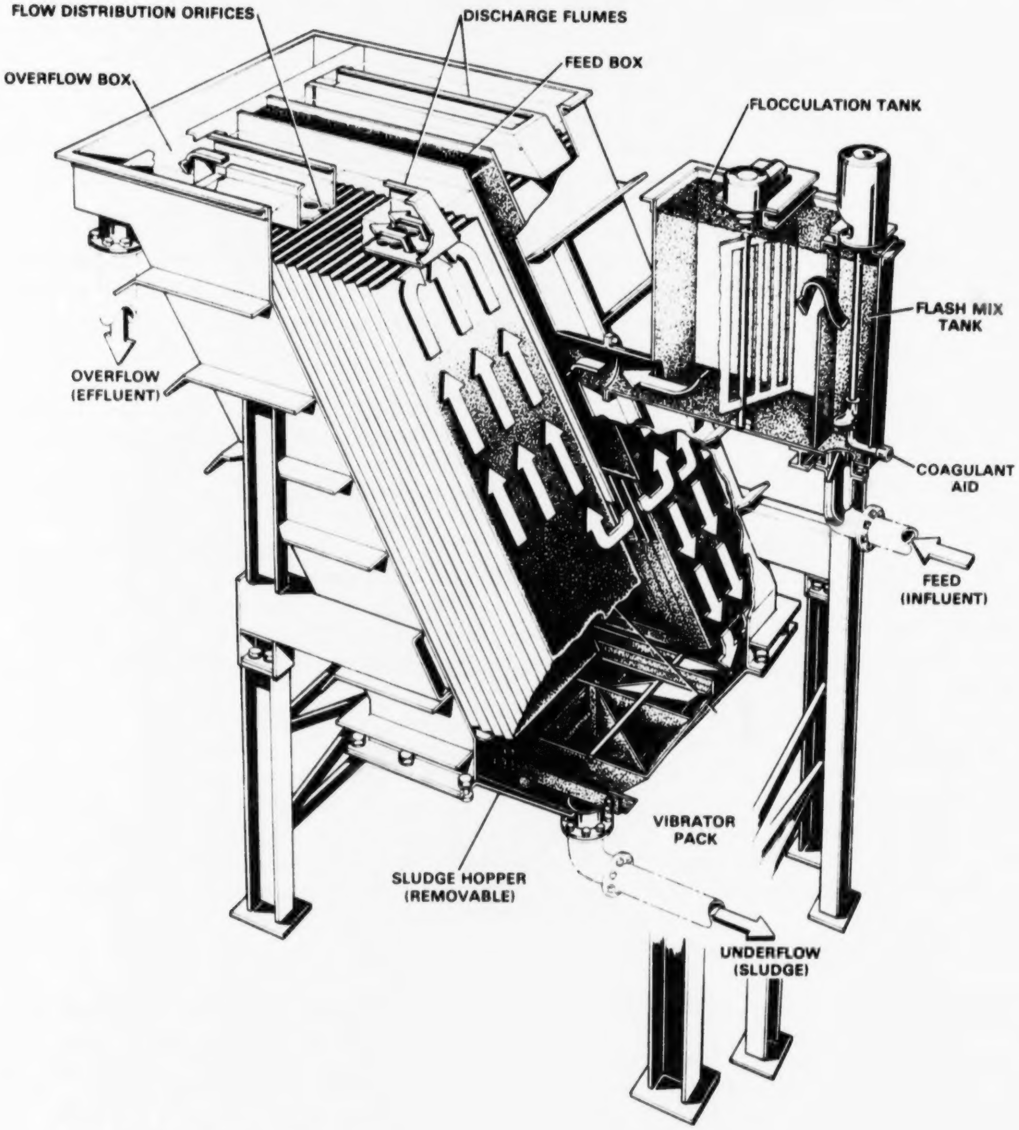
High rate clarifiers are not suitable for removal of particles larger than 0.1 inch or less than 10 microns. Use of high rate gravity settlers has not been demonstrated for applications in solid/water separation and they are generally used in applications with lower solids concentrations (Mallory and Nawrocki, 1974). There is the possibility that cohesive sediments or soils may clog the channels, tubes, or plates (Jones, Williams and Moore, 1978).

d. Technology Selection/Evaluation

Sedimentation employing impoundment basins and conventional clarifiers is a well established technology for removing particles ranging in size from gravel down to fine silt. However, proper flocculation is essential to ensure removal of silt-sized paticles. Sedimentation methods have not been widely employed for classifying solids according to particle-size. They can be expected to be less effective in classifying solids than other methods described in this section (e.g., classifier, cyclones, and screens).

Impoundment basins have a high capital and operating cost. For this reason their use is generally limited to large-scale cleanup operations.

FIGURE 10-20. HIGH RATE GRAVITY SETTLER



Source: Parkson Corp., 1984

Impoundment basins also pose the greatest potential for secondary impacts of all solids separation methods; contaminants may leach into groundwater if the liner system is not properly designed and the large surface area of the impoundment can result in volatilization of contaminants and localized air pollution problems.

Impoundment basins require a long set-up time and the need to obtain construction permits can further delay the start-up of the cleanup operation. Conventional clarifiers and high-rate clarifiers eliminate some of the problems associated with impoundment basins. Operating costs are substantially less because clarifiers have a build-in solids collection system. Clarifiers also pose no threat to groundwater contamination. However, capital costs associated with the use of clarifiers can also be quite high for a large-scale cleanup operation.

Both types of clarifiers can be barge mounted in areas of limited space. High-rate clarifiers with their relatively small space requirements, may be the only suitable sedimentation method in congested areas. Clarifiers and impoundment basins are easy to operate and maintain.

10.2.2 Dewatering

Dewatering is a physical unit operation used to reduce the moisture content of slurries or sludges in order to facilitate handling and prepare the materials for final treatment or disposal. Devices which can be used to dewater slurries or sludges include gravity thickeners, centrifuges, filters, and dewatering lagoons. Selection of the most appropriate method depends on such factors as the volume of the slurry, solids content of the waste stream, land space availability and the degree of dewatering required prior to treatment or disposal.

Although several of the dewatering methods are extremely effective in removing water, the solids are often not sufficiently dry to meet requirements for final disposal, and require further treatment to fixate or solidify the wastes (Section 10.3). The contaminated water generated during dewatering generally contains hazardous constituents as well as several hundred to several thousand mg/l suspended solids, and will require additional treatment (Section 10.1).

10.2.2.1 Gravity Thickening

a. General Description

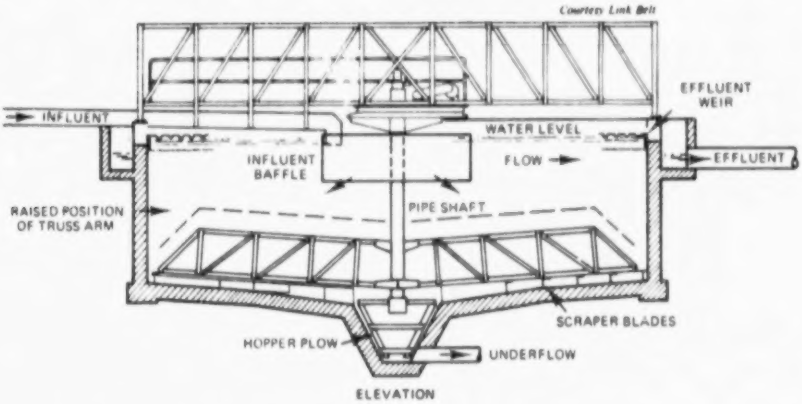
Gravity thickening is generally accomplished in a circular tank, similar in design to a conventional clarifier. The slurry enters the thickener through a center feedwell designed to dissipate the velocity and stabilize the density currents of the incoming stream (Figure 10-21). The feed sludge is allowed to thicken and compact by gravity settling. A sludge blanket is maintained on the bottom to help concentrate the sludge. The clarified liquid overflows the tank and the underflow solids are raked to the center of the tank and withdrawn by gravity discharge or pumping. Flocculants are often added to the feed stream to enhance agglomeration of the solids and promote quicker or more effective settling (Metcalf and Eddy, 1979; Dorr-Oliver, 1981). Tanks are usually constructed of concrete or steel.

Gravity thickener size and specifications depend on the following factors: maximum flow, type of wastes, pH, volume of solids/day, percent solids, specific gravity, maximum particle size, and percent solids required in the underflow.

b. Applications/Limitations

Gravity thickeners are used to concentrate slurries and are capable of achieving a solids concentration of approximately 2 to 15 percent (USEPA, 1979). They generally produce the thinnest and least concentrated sludge of all the dewatering methods described in this section. The intent in using a gravity thickener is usually to reduce the hydraulic load of a slurry that is to be fed to a more efficient dewatering method, such as filtration or centrifugation. They also provide a high sludge storage capacity. Conventional gravity thickeners require large land areas for operation and therefore are

FIGURE 10-21. GRAVITY THICKENER



Source: Culp, Wesner, and Culp, 1978

not applicable where space is restricted. However, high-rate gravity thickeners designed to provide up to 15 times the throughput of a conventional thickener are available and can reduce land requirements considerably (Dorr-Oliver, 1981).

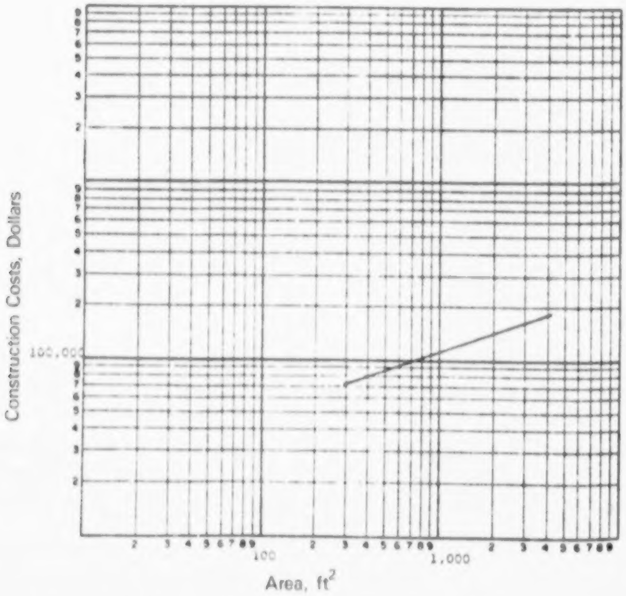
c. Technology Selection/Evaluation

Gravity thickening provides a simple, low maintenance method for concentrating slurries, thereby reducing the hydraulic load to subsequent dewatering processes. They are suitable for operations where a high degree of operator supervision cannot be provided. Because of the requirements for a large surface area, localized air pollution and odors may be significant.

d. Costs

Equipment costs for gravity thickeners are illustrated in Figure 10-22. Costs are based on the use of a circular reinforced concrete basin and related drive and motor.

FIGURE 10-22. GRAVITY THICKENING CONSTRUCTION COSTS, 1975*



***Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.908)**

Source: Culp, Wesner, and Culp, 1978

10.2.2.2 Dewatering Lagoons

a. General Description

Dewatering lagoons use a gravity or vacuum assisted underdrainage system to remove water. The base of the lagoon is lined with clay plus a synthetic liner or other appropriate liner material to prevent migration of contaminants into the underlying soils and groundwater. At a minimum, the liner consists of a low permeability clay layer which is several feet thick. When the lagoon is no longer in use, the clay liner is excavated and properly disposed of. In some instances this design may not be adequate to protect groundwater supplies. A combination clay/synthetic liner and a secondary leachate collection system are required in some instances.

The underdrainage system can be designed and operated using one of the following approaches:

- Gravity underdrainage - This system consists of a filler material (well-graded sand or filter fabric) underlain by a porous free-draining gravel layer. Perforated drainage pipe is embedded in the gravel. The drainage pipe network is designed with flow gradients leading towards a central collection point or sump. Information on the hydraulic design of a gravity drainage system can be found in Section 5.2.

- Vacuum pumping - Vacuum pumping systems can use either pumped wells or wellpoints. Pumped wells with large vacuum pumps may be installed directly in the waste material. Wellpoints may be used, provided they are installed to the depth of an underlying sand filter. Installation of wellpoints directly in the sludge, contaminated sediments, or soils is not cost-effective, because it is necessary to space the wellpoints very close together in order to dewater low permeability material (Haliburton, 1978). Information on the design of wells and wellpoints can be found in Section 5.1.

- Vacuum assisted drying beds - Vacuum assisted drying beds use a porous media filter plate set above an aggregate filled support plenum which drains to a sump. A relatively small vacuum pump is connected to drain a vacuum from the sump. The vacuum is activated when the volume of the slurry has been reduced by half due to gravity drainage. The vacuum holds until the solids crack, allowing air through the bed (USEPA, 1982).

- Electroosmosis - This technique involves a process in which a direct current electrical potential is set-up in the soil by means of electrodes. This electric potential induces the flow of water in the pores of the fine-grained sediment or sludge towards the negative pole, or cathode. A line of wells or wellpoints can be installed to intercept and remove the water (Mallory and Nawrocki, 1974).

b. Applications/Limitations

Dewatering lagoons are best suited to large-scale dewatering operations where the volume of sludge or sediment would require an inordinately large number of mechanical dewatering units (e.g., filters or centrifuges). Lagoons are one of the more effective dewatering methods. A gravity dewatering system is capable of achieving 99 percent solids removal and a dewatered cake of 35 to 40 percent solids after 10 to 15 days (based on municipal sludges) (DeRenzo, 1978; USEPA, 1982). Vacuum assisted systems may be capable of achieving a dry cake in a shorter retention time.

The major limitations on the use of dewatering lagoons is that they require large land areas and long set-up times. Because of their large surface area they may not be well suited to areas with heavy rainfall or to areas where long periods of freezing would prevent dewatering.

Each of the types of dewatering lagoons described in this section has its own specific applications and limitations.

Gravity drainage systems have the lowest operating costs. However, dewatering is achieved at a relatively slow rate and this may result in the need for more land area than required with the other methods. Gravity drainage systems are also more prone to clogging, particularly if the system is not carefully designed.

Vacuum pumping or vacuum assisted dewatering beds are capable of dewatering at a much more rapid rate than gravity systems. Vacuum assisted dewatering beds reportedly increase the rate of dewatering by about 50 percent (with a negative pressure of 8 psi or less) (Haliburton, 1978). However, they require a higher degree of maintenance and are considerably more costly to operate than gravity systems.

Electroosmosis is a very costly technique which would be limited to dewatering of very fine grained (2 to 10 microns), very hazardous and difficult to dewater solids.

c. Technology Selection/Evaluation

Dewatering lagoons provide an effective means of dewatering solids. They are also versatile in that they can provide storage capacity for solids prior to disposal. Of all the dewatering technologies they require the largest time to implement and have the greatest potential for secondary impacts due to localized air pollution and groundwater contamination. Operating costs are higher than other dewatering technologies because of the need to remove the solids with mechanical dredging equipment.

10.2.2.3 Centrifuges

a. General Description

Centrifugal dewatering is a process which uses the force developed by fast rotation of a cylindrical drum or bowl to separate solids and liquids by density differences under the influence of centrifugal force. Dewatering is usually accomplished using solid bowl or basket centrifuges. Disc centrifuges are also available and are mainly used for clarification and thickening. Figures 10-23 through 10-25 illustrate the three types of centrifuges.

The operation of the solid-bowl centrifuge is a continuous process. The unit consists of a long bowl, normally mounted horizontally and tapered at one end. Sludge is introduced into the unit continously and the solids concentrate on the periphery. A helical scroll within the bowl, spinning at a slightly different speed, moves the accumulated sludge towards the tapered end where additional solids concentration occurs prior to discharging the solids (USEPA, 1982a and USEPA, 1979).

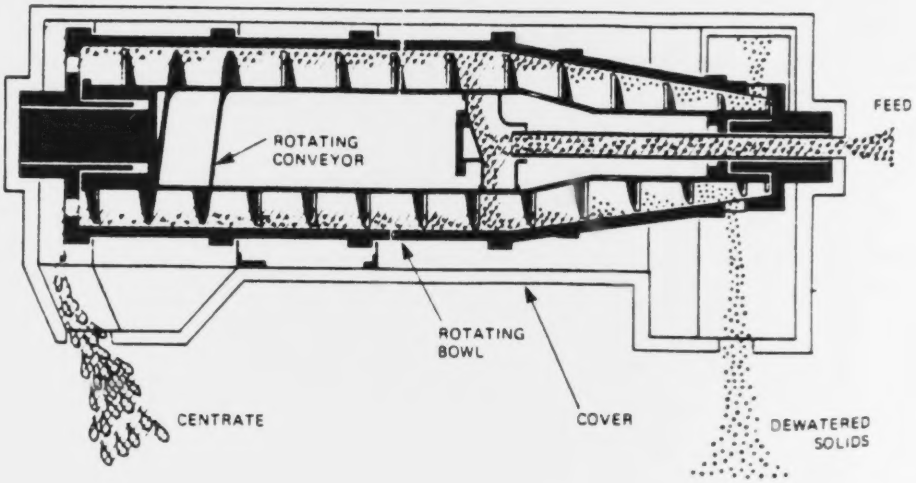
In the basket centrifuge, flow enters the machine at the bottom and is directed toward the outer wall of the basket. Cake continually builds up within the basket until the centrate, which overflows a weir at the top of this unit, begins to increase in solids. At that point, feed to the unit is shut off, the machine decelerates, and a skimmer enters the bowl to remove the liquid layer remaining in the unit. A knife is then moved into the bowl to cut out the cake which falls out the open bottom of the machine. The unit is a batch device with alternate charging of feed sludge and discharging of dewatered cake (USEPA, 1982a and USEPA, 1978).

In the disc centrifuge, the incoming stream is distributed between a multitude of narrow channels formed by stacked conical discs. Suspended particles have only a short distance to settle, so that small and low density particles are readily collected and discharged continuously through fairly small orifices in the bowl wall. The clarification capability and throughput range are high, but sludge concentration is limited by the necessity of discharging through orifices of 0.05 inches to 0.1 inches in diameter. Therefore, it is generally considered a thickener rather than a dewatering device (USEPA, 1978).

b. Applications/Limitations

Centrifuges can be used to concentrate or dewater soils and sediments ranging in size from fine gravel down to silt. Effectiveness of centrifugation depends upon the particle sizes and shapes, and the solids concentration among other factors. Data from the dewatering of municipal sludges (where extensive information is available), indicate that solids concentrations ranging from about 15 to 40 percent are achievable with the

FIGURE 10-23.
SCHEMATIC OF TYPICAL SOLID BOWL DECANTER CENTRIFUGE



Source: USEPA, 1979

FIGURE 10-24.
GENERAL SCHEMATIC OF IMPERFORATE BASKET CENTRIFUGE

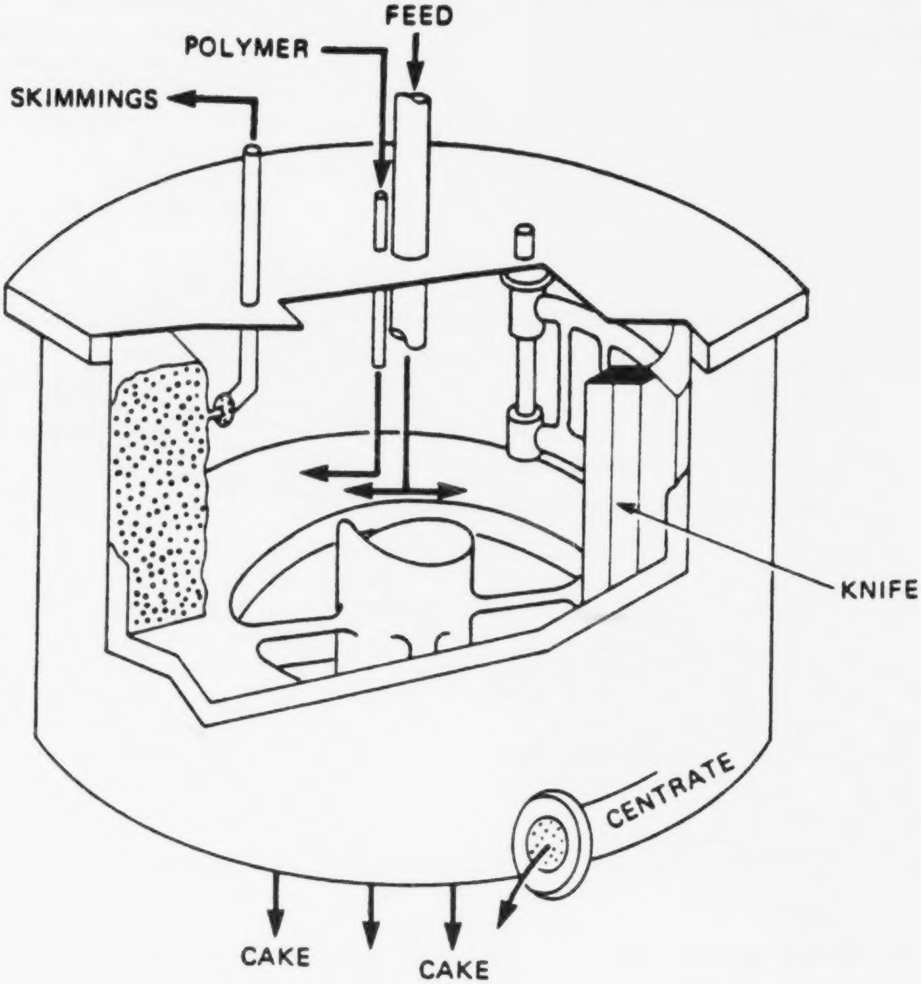
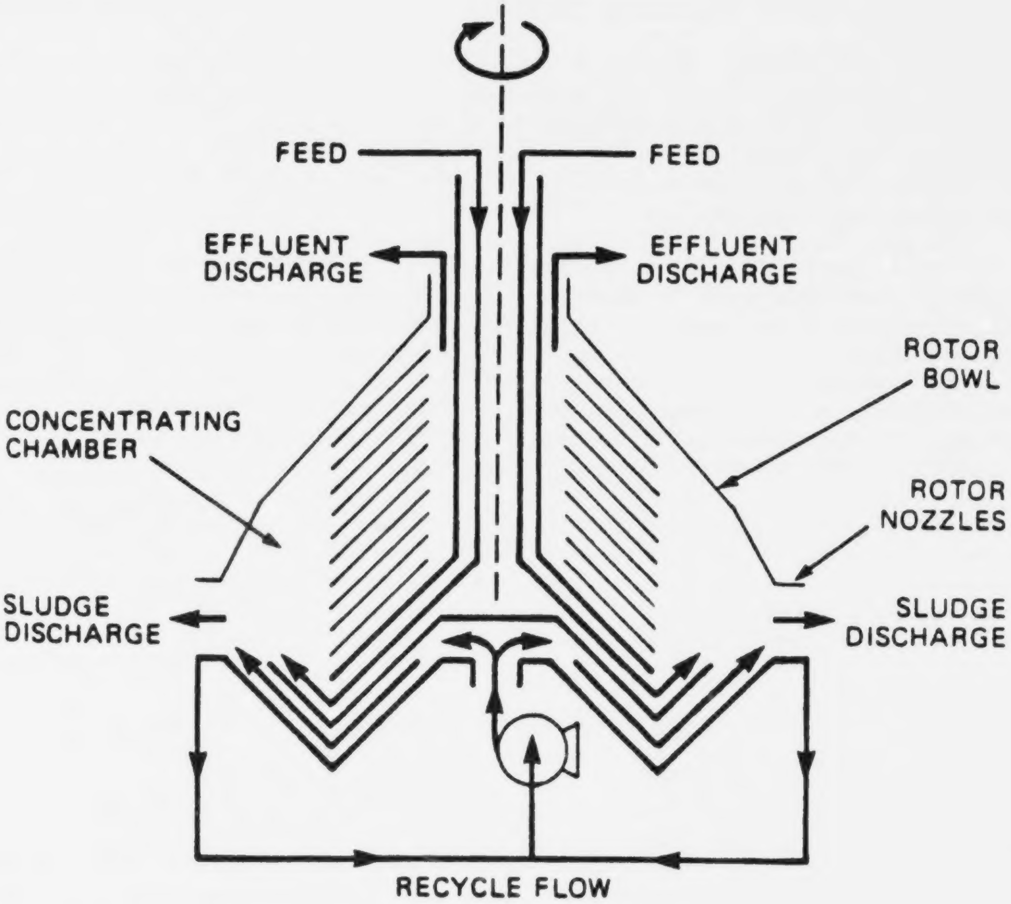


FIGURE 10-25.
SCHEMATIC OF A DISC NOZZLE CENTRIFUGE



Source: USEPA, 1979

solid bowl centrifuge. For the basket centrifuges the cake solids concentration typically ranges from about 9 to 25 percent. Solids capture typically ranges from about 85 to 97 percent with chemical conditioning, both for the solid bowl centrifuge and the basket centrifuge. Disc centrifuges can concentrate a 1 percent sludge to a 6 percent solids (USEPA, 1978; USEPA 1982a).

Centrifuges are capable of removing particles as small as 1 micron in diameter. However, removal efficiencies are reduced dramatically for particles smaller than 10 microns (Krizek, Fitzpatrick and Atmatzidis, 1976).

Although the basket centrifuge does not acheive as dry a cake as does the solid bowl centrifuge, it has the advantage of being able to handle hard to dewater sludges and is not significantly affected by grit. It has the highest capital cost but lowest operation and maintenance cost of the three centrifuge types (USEPA, 1979 and USEPA, 1982b). A major limitation is that it must be operated on a batch basis.

The solid bowl centrifuge is very flexible in that it can handle higher than design loadings, such as temporary increases in hydraulic loading or solids concentrations; however, the cake solids content may be reduced. Higher feed rates make the solid bowl centrifuge better suited for large-scale dewatering operations. Maintenance and pretreatment requirements are more extensive than for the basket centrifuge. The scroll of the solid bowl centrifuge is very susceptable to abrasion. This results in the need to degrit the effluent (USEPA, 1979 and USEPA, 1982a).

The disc centrifuge has more limited application at hazardous waste sites than the other types of centrifuges. Although it can yield a highly clarified centrate even without the use of chemicals, the percent solids concentration is low, maintenance requirements are relatively high, and pretreatment requirements (grit and fibrous material removal) are extensive.

c. Technology Selection/Evaluation

Centrifugation offers a simple, clean and reliable method for dewatering sludges and other solids. They are less effective than filtration methods and dewatering lagoons, but more effective than gravity thickeners. Centrifuges are compact and are well suited to use in mobile treatment systems.

Although reliable for their intended function, centrifuges generated a centrate and a sludge which require further treatment prior to disposal. Suspended solids levels from centrifugation may be as high as several thousand parts per million.

Since centrifugation relies on the settling of particles according to density, the process tends to classify the solids, settling the heavier particles first. Dewatering processes which rely on filtration achieve a more even distribution of solid capture. It is possible for a buildup of fines to occur in the effluent from centrifugation, particularly if the centrifuge is operating improperly due to inadequate solids conditioning or due to a malfunction (USEPA, 1982a). Since most organic and inorganic waste constituents tend to sorb to fine clay and silt particles, this may result in unacceptable levels of contaminants in the overflow.

Advantages of centrifuges for thickening and dewatering methods include relatively limited space requirements and fast start-up and shut-down. They also generate little or no air emissions since the process is essentially enclosed.

d. Costs

This section presents construction and O&M costs for basket and solid bowl centrifuges. It should be recognized that the curve for construction cost is not capital cost. The curve does not include costs for special site work, general contractor overhead and profit, engineering, land, legal, fiscal, and administrative work and interest during construction. These cost items are all more directly related to the total cost of a project rather than the cost of any one of the individual unit processes. These costs are therefore most appropriately added following cost summation of the individual unit processes, if more than one unit process is required. Typically, these costs add 35 to 45 percent, depending on project size and complexity, to the actual construction costs which are shown in the curves (USEDA, 1982a).

Construction costs include housing for the centrifuges. Housing costs may not be applicable for hazardous waste sites because of the short time period the unit will be on-site.

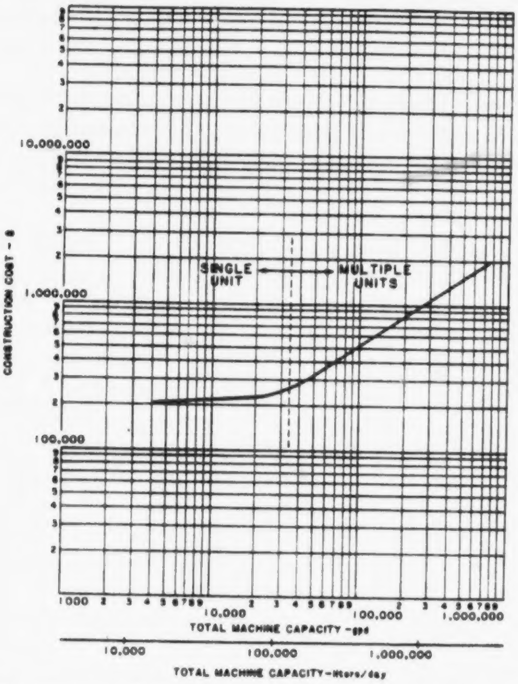
No costs were available for mobile treatment units involving use of centrifuges.

Basket Centrifuge -- Figure 10-26 shows construction costs for single and multiple basket centrifuges with capacities ranging from 4,000 to 700,000 gpd. Centrifuge costs are for automatic machines operating on a preprogrammed cycle, an approach which requires only minimal operator attention.

In addition to the basic machines, the costs include equipment for polymer preparation, storage, and application. If other conditioning chemicals are used, the costs would have to be adjusted accordingly. The costs do not include sludge and centrate pumping, sludge conveying, and sludge storage. It was assumed that centrifuges are located in two story concrete block buildings with bottom discharge to trucks or storage bins. Housing requirements were developed from equipment manufacturers' recommended layouts (USEPA, 1982a).

Figures 10-27 and 10-28 present O&M costs for basket centrifuges. Electrical energy requirements were computed from connected and operating

FIGURE 10-26. CONSTRUCTION COST FOR BASKET CENTRIFUGES *



***Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)**

Source: USEPA 1982a

horsepower information provided by equipment manufacturers. Basket centrifuge operating horsepower, computed on the basis of a complete cycle involving machine acceleration, sludge feeding, skimming, decelerating, and sludge plowing, averages 40 to 60 percent of the connected horsepower. Electrical power for polymer preparation and feeding is included, but energy for sludge pumps, centrate pumping and sludge conveying equipment is not included.

Maintenance costs were obtained from equipment manufacturers and from operating installations and represent an industrywide average of annual expenditures for maintenance, replacement parts, lubrication, and other consumable items associated with basket centrifuge operation. Maintenance material costs do not include the cost of polymers.

Labor requirements for O&M assume 24 hours per day of operation, with occasional downtime for maintenance as required. The major portion of the operating labor is devoted to machine start-up and adjustment, polymer preparation, and required maintenance (USEPA, 1982a).

FIGURE 10-27. BASKET CENTRIFUGES—BUILDING ENERGY, PROCESS ENERGY AND MAINTENANCE MATERIAL REQUIREMENTS*

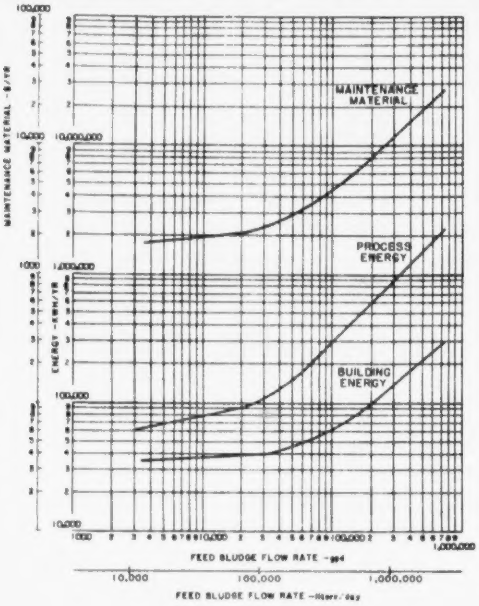
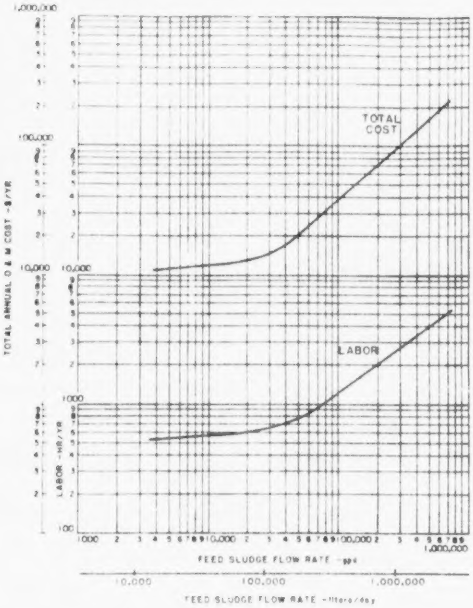


FIGURE 10-28. BASKET CENTRIFUGES—LABOR AND TOTAL ANNUAL OPERATION AND MAINTENANCE COST*



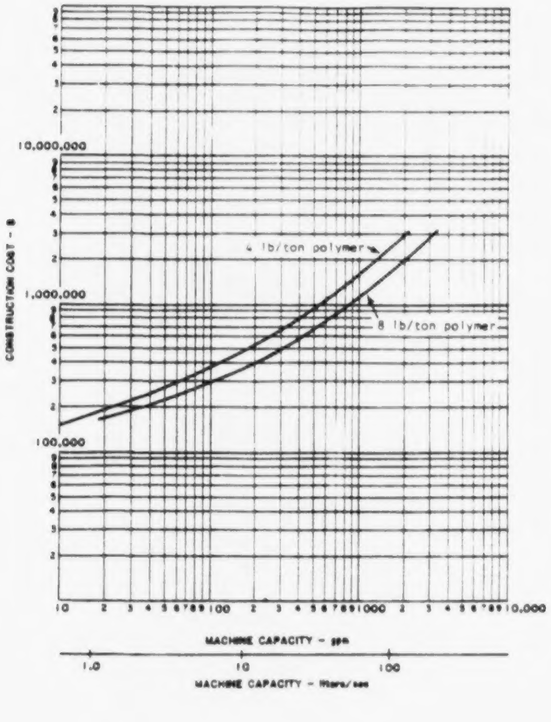
*Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)

Source: USEPA, 1982a

It should be recognized that operation and maintenance costs will vary widely depending on sludge dewatering characteristics and specific operating conditions related to the installation, and appropriate adjustment should be made if conditions vary significantly from those stated above (USEPA, 1982a).

Solid Bowl Centrifuge (High G)--Construction costs for solid bowl centrifuges are shown in Figure 10-29. Machine throughput is significantly affected by the polymer dosage, and therefore the construction cost for a given feed rate varies with the polymer dose. In this figure, single machines were assumed to be used for feed rates up to 500 gpm, with multiple units being usedfor higher feed rates. All machines are equipped with automatically controlled eddy current backdrive and have sintered tungsten carbide conveyor tips. Polymer storage preparation, and feed equipment is included in the costs, but costs for sludge feed pumping and centrate pumping are not included (USEPA, 1982a).

FIGURE 10-29. CONSTRUCTION COST FOR A HIGH G SOLID BOWL CENTRIFUGES*



*Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)

Source: USEPA, 1982a

Operation and Maintenance Cost for solid bowl centrifuges are shown in Figure 10-30 and 10-31. Process energy was calculated from information supplied by a manufacturer of high G centrifuges and assumes use of an eddy current backdrive. Energy requirements could be reduced between 5 to 20 percent if the backdrive is not utilized. Included in the process energy requirements are the main drive motor, the eddy current backdrive, and equipment required for polymer preparation and feed. Energy required for feed sludge pumping and handling of the dewatered sludge is not included.

Maintenance material requirements include replacement of the conveyor tips every 30,000 hours of operation, as well as replacement of other necessary components of the centrifuge and the electrical controls.

Operation and maintenance labor requirements are based on 24 hours per day of continuous operation. Most operational labor is devoted to polymer preparation and machine start-up and adjustment. Occasional maintenance is required for lubrication, with more extensive maintenance required approximately every 30,000 hours for replacement of the sintered tungsten carbide conveyor tips.

The cost curves presented do not include the cost of polymer. The polymer dosage is highly dependent on the characteristics of the sludge being dewatered, and polymer dosage will also have a great influence on the throughput of the centrifuge.

10.2.2.4 Filtration

a. General Description

Filtration is a physical process whereby particles suspended in a fluid are separated from it by forcing the fluid through a porous medium. Three types of filtration are commonly used for dewatering: belt press filtration, vacuum filtration, and pressure filtration.

Belt filter presses employ single or double moving belts to continuously dewater sludges. As Figure 10-32 illustrates, the belt press filtration process includes three stages: chemical conditioning of the feed, gravity drainage to a nonfluid consistency and dewatering. A flocculant is added prior to feeding the slurry to the belt press. In the next step, free water drains from the conditioned sludge. The sludge then enters a two-belt contact zone, where a second upper belt is gently set on the forming sludge cake. The belts with the captured cake between them pass through rollers of generally decreasing diameter. This stage subjects the sludge to continuously increasing pressures and shear forces. Progressively more and more water is expelled throughout the roller section to the end where the cake is discharged. A scraper blade is often employed for each belt at the discharge point to remove the cake from the belts (USEPA, 1982).

FIGURE 10-30. HIGH G SOLID BOWL CENTRIFUGES—BUILDING ENERGY, PROCESS ENERGY AND MAINTENANCE MATERIAL REQUIREMENTS*

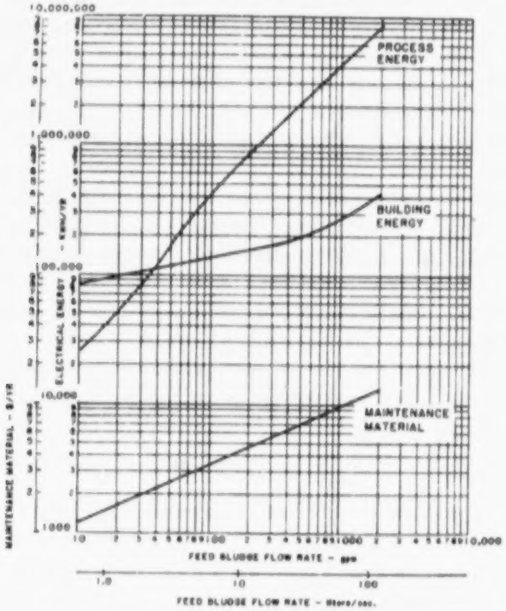
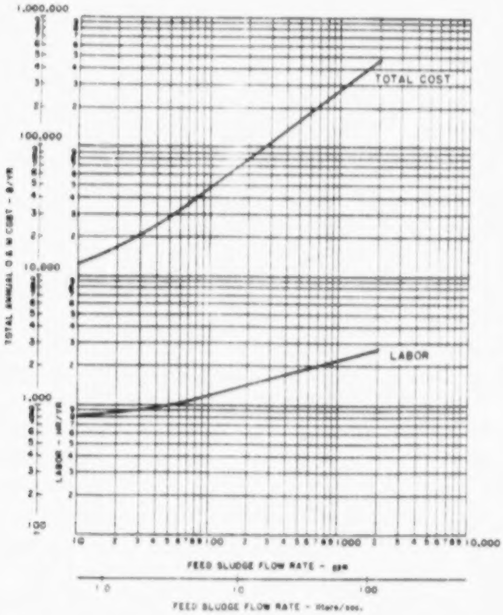


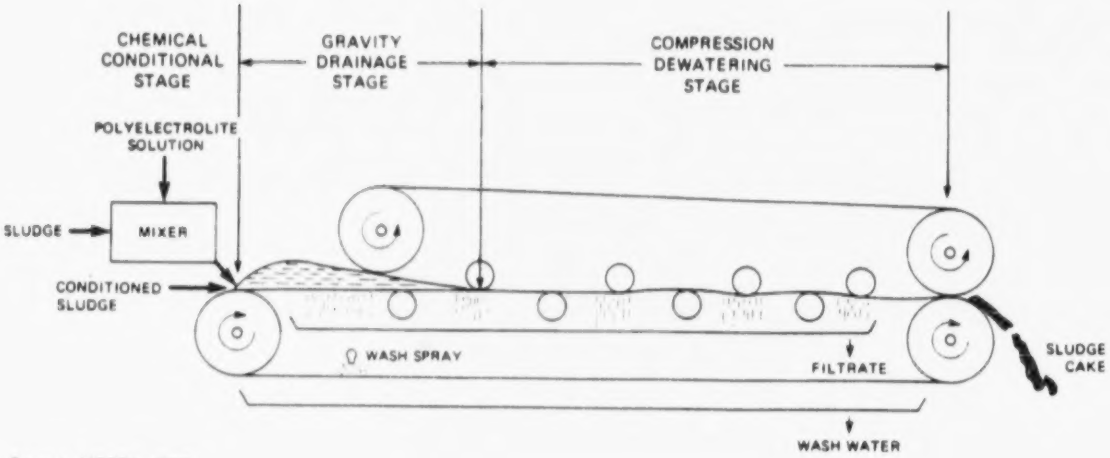
FIGURE 10-31. HIGH G SOLID BOWL CENTRIFUGES—LABOR AND TOTAL ANNUAL OPERATION AND MAINTENANCE COST*



*Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)

Source: USEPA 1982a

FIGURE 10-32. THE THREE BASIC STAGES OF A BELT PRESS

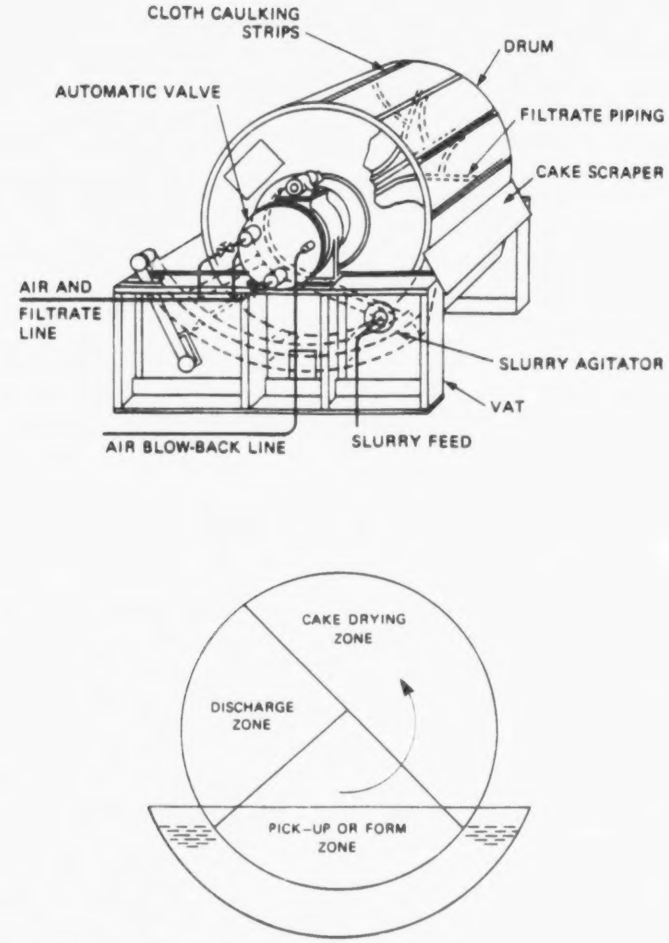


Source: USEPA, 1979

A vacuum filter consists of a horizontal cylindrical drum which rotates partially submerged in a vat of sludge (Figure 10-33). The drum is covered with a continuous belt of fabric or wire mesh. A vacuum is applied to the inside of the drum by means of a connection within a central trunion. The vacuum causes liquid in the vat to be forced through the filter medium leaving wet solids adhering to the outer surface. As the drum continues to rotate, it passes from the cake forming zone to a drying zone, and finally to a cake discharge zone where the sludge cake is removed from the media (Metcalf and Eddy, 1979; USEPA, 1982).

Pressure filtration is used to describe a category of filters in which rigid individual filtration chambers are operated in parallel under relatively high pressure. The filter press (Figure 10-34), the most common representative of the group consists of vertical plates that are held rigidly in a frame and are pressed together by a large screw jack or hydraulic cylinder as shown in Figure 10-34. The liquid to be filtered enters the cavity formed by the frame. Pressed against this hollow frame are perforated metal plates covered with fabric filter medium. The plate operates on a cycle which includes filling, pressing, cake removal, media washing, and press closing (USEPA, 1982a and USEPA, 1979). As the liquid flows through the filter

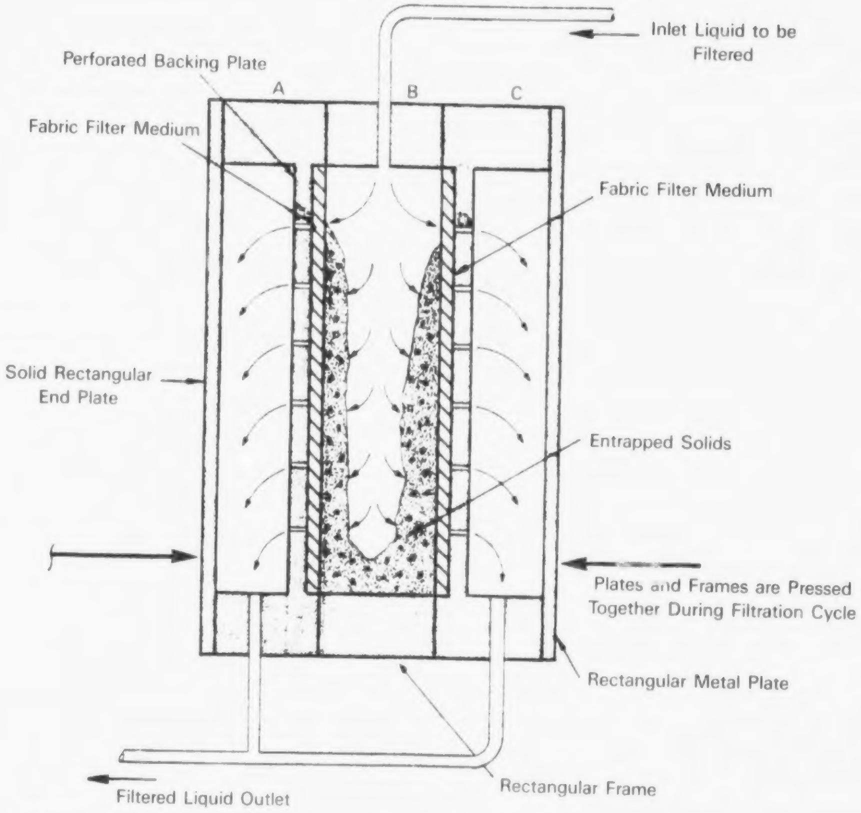
FIGURE 10-33.
ROTARY VACUUM FILTER



Source: USEPA, 1979

FIGURE 10-34.

FILTER PRESS (ILLUSTRATIVE CROSS-SECTIONAL VIEW OF ONE RECTANGULAR CHAMBER)



When the cavity formed between plates A and C is filled with solids, the plates are separated.
The solids are than removed and the medium is washed clean.
The plates are than pressed together and filtration resumed.

Source: De Renzo, 1978

medium, solids are entrapped and buildup within the cavity until the cavity is full. The slurry is dewatered until no filtrate is produced. The press is then opened, the dewatered slurry is removed, the plates are cleaned, and the cycle is repeated (De Renzo, 1978). In certain applications, the filter media is precoated with diatomaceous earth, fly ash, or other filter aids to improve performance.

Diaphragm filters are specially designed filter presses. Instead of the conventional plate and frame unit in which constant pumping pressure is used to force the filtrate through the cloth, diaphragm filters combine an initial pumping followed by a squeezing cycle that can reduce the cost and process time.

b. Applications/Limitations

Filtration can be used to dewater solids over a wide range of solids concentrations and particle sizes. Effectiveness for a particular application depends on the type of filter, the particle size distribution, and the solids concentrations. For dewatering of municipal sludges where considerable performance data is available, typical ranges for solids content and solids removal or capture are as follows (USEPA, 1979; USEPA, 1982; Metcalf and Eddy, 1979):

	Solids Content (%)	Solids Capture (%)
Belt Press Filtration	15 to 45	85 to 95
Vacuum Rotary Filtration	15 to 35 or 40	88 to 95
Pressure Filter	30 to 50	98

Manufacturers' data is also available on the performance of filtration methods in dewatering coal slurries. This data indicates that belt press and filter press filtration are capable of producing a filter cake of up to 70 to 80 percent solids. Also, tests conducted by Rexnord, Inc. demonstrated that high density dredged materials can be dewatered to a cake solids concentration of 70 percent using belt press filtration (Erickson and Hurst, 1983).

Although the filter press achieves a dry filter cake and has the greatest capacity for solids capture, there are a number of other factors which enter into the decision to use a particular method of filtration.

Filter presses generally require larger quantities of conditioning chemicals than the other filtration methods. They also have the highest capital and operating cost and require the largest amount of space. Replacement of filter media on a filter press is both expensive and time consuming (USEPA, 1982a).

Vacuum filtration is the most energy intensive of the three methods and the least effective in dewatering. Another limitation on the use of vacuum filtration is that the incoming feed must have a solids content of at least 3 percent in order to achieve adequate cake formation (USEPA, 1982a). A big advantage to vacuum filtration is that because dewatering is accomplished by a vacuum rather than by mechanical means, the hydraulic throughput is higher than for the other filter types. Vacuum filtration has an advantage over belt press filtration in that it is easier to maintain and can operate effectively even without optimum chemical conditioning.

Recent advances in belt press filtration has made this method nearly as effective as pressure filtration. The belt press filter also has the advantage of being the least energy intensive of the filtration methods. The major limitation on the use of this method is that the process is very sensitive to incoming feed characteristics and chemical conditioning. However, these limitations can be overcome to a certain extent; most belt presses can be equipped with sensing devices which can be set to automatically shut off feed flow in the case of underconditioning. The feed characteristics can be optimized by carefully prescreening the slurry to remove large objects and fibrous material which can deteriorate the belt quickly (USEPA, 1982a).

c. Technology Selection/Evaluation

Filtration appears to offer the most effective method for dewatering slurries. The processes are generally reliable, provided the slurries have been properly prescreened and conditioned. Filtration equipment, particularly belt press and vacuum filtration, is well suited for inclusion in mobile treatment systems. Mobile systems are available from several manufacturers. However, the maintenance requirements associated with filtration are significant. The filter cloth or belts must be periodically replaced and the filter media periodically washed to remove contaminated solids.

Despite their effectiveness in dewatering sludges, both the filtrate and the dewatered sludge are likely to require further treatment prior to disposal. The water generated from washing of the filter media will also require treatment.

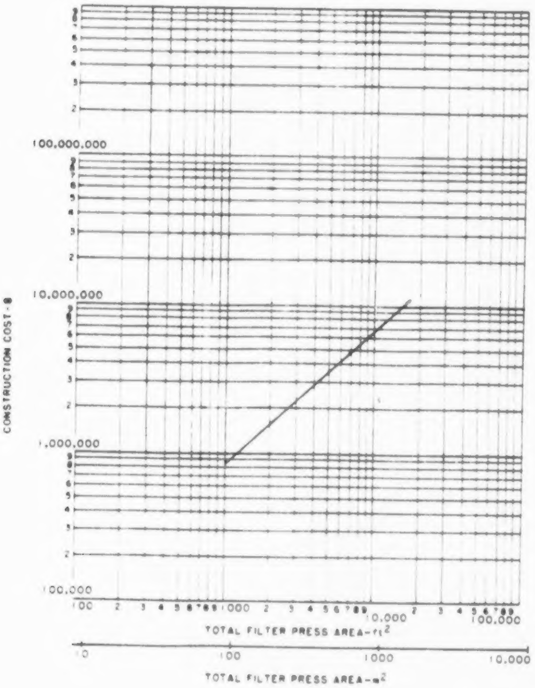
d. Costs

This section presents construction and O&M costs for diaphragm, belt-press and vacuum filters. It should be recognized that the curve for construction cost is not capital cost. The curve does not include costs for special site work, general contractor overhead and profit, engineering, land, legal, fiscal, and administrative work and interest during construction. These cost items are all more directly related to the total cost of a project rather than the cost of any one of the individual unit processes. These costs are therefore most appropriately added following cost summation of the individual unit processes, if more than one unit process is required. Typically, these costs add 35 to 45 percent, depending on project size and complexity, to the actual construction costs which are shown in the curves (USEPA 1982a).

Diaphragm filter press--Construction costs for diaphragm filter presses ranging in size from 1,200 to 15,505 ft^2 are shown in Figure 10-35. The largest machine manufactured is about 6,000 ft^2, and multiple presses are required for larger press areas. Construction costs include the diaphragm press, feed pump, pumps for the diaphragm and cloth washing, vacuum pumps an air compressor and receiver, lime and ferric chloride storage and feed facilities and all electrical and controls necessary for complete automatic operations. Housing costs are also included (USEPA, 1982a).

Operation and maintenance costs shown in Figures 10-36 and 10-37 were developed for a 4 percent feed of anaerobically digested sludge, chemically conditioned with 5 percent ferric chloride and a 20 percent lime. Press loading was 1.0 lb/sq ft/hr, without chemicals, and cake discharge was taken at 35 percent. Press operation time was 19 hours per day, with the remaining time dedicated to press cleanup and maintenance.

FIGURE 10-35. CONSTRUCTION COST FOR DIAPHRAGM FILTER PRESS*



***Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)**

Source: USEPA, 1982a

FIGURE 10-36. DIAPHRAGM FILTER PRESS–LABOR AND TOTAL ANNUAL OPERATION AND MAINTENANCE COST*

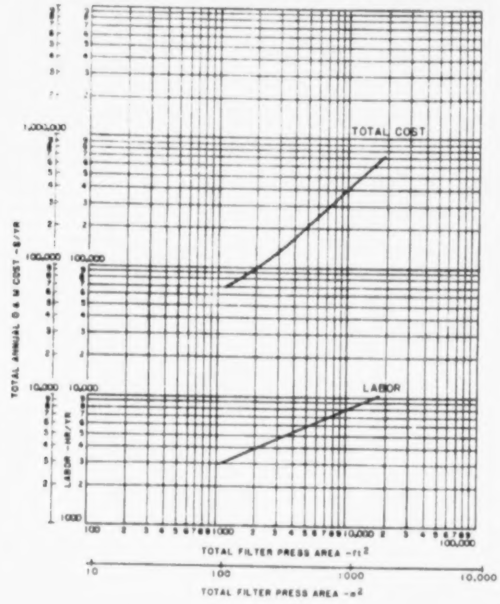
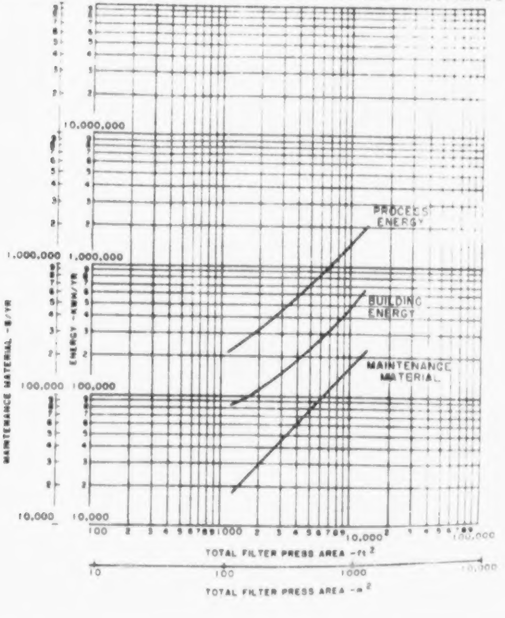


FIGURE 10-37. DIAPHRAGM FILTER PRESS–BUILDING ENERGY, PROCESS ENERGY AND MAINTENANCE MATERIAL REQUIREMENTS*



***Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)**

Source: USEPA, 1982a

Process energy requirements are for the sludge feed pump, the air pump for inflating the diaphragm, and a vacuum pump for removal of liquid sludge remaining in the internal piping prior to opening the press. Energy is also required to open and close the press, for cloth washing, and for conditioning chemical preparation and feed. Building energy requirements are based on 84 kwh/sq ft/yr.

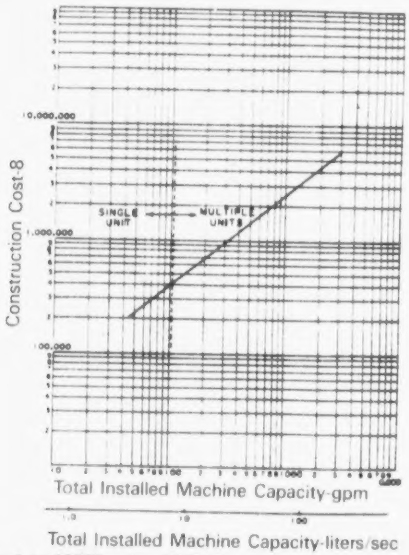
Maintenance material costs consist principally (over 90 percent) of replacement of diaphragms and filter cloths. Other costs are for miscellaneous equipment parts and for miscellaneous electrical components.

Labor required is for both operation and maintenance, with the majority of the labor devoted to operational requirements. Labor requirements are based on operational experience of the manufacturer (USEPA, 1982a).

Belt Press Filters--Construction costs are for belt filter press dewatering systems that include the belt press unit, wash water pump, conditioning tank, feed pump, polymer storage tank and pump, belt conveyor, and electrical control panel. Machines are generally sized using metric dimensions and are rated on the basis of sludge flow in gpm/m of belt width. For mixtures of digested primary and secondary sludges, a value of 50 gpm/m belt width is a typical loading recommendation, and was used in the cost development. HIgher loadings are possible in some cases if the sludge can be easily dewatered (USEPA, 1982a).

Estimated construction costs are presented in Figure 10-38 as a function of total installed machine capacity.

FIGURE 10-38. CONSTRUCTION COST OF A BELT FILTER PRESS



***Costs can be updated to \$1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)**

Source: USEPA, 1982a

Figures 10-39 and 10-40 present operation and maintenance costs for the belt filter press. Process energy requirements were developed from the total connected horsepower for the belt drive unit, belt wash water pump, conditioning tank, feed pump, polymer pump and tanks, belt conveyor, and electrical control panel. A belt filter loading of 50 gpm/m of machine width was used in selecting unit sizes and determining power requirements. Twenty-two hours of continuous operation with 2 hour of downtime for routine maintenance was assumed in calculating process energy requirements (USEPA, 1982a).

Labor and maintenance requirements were estimated from information provided by equipment manufacturers, as well as information from plants operating belt filter presses. The maintenance material requirments assume the replacement of a set of belts every 6 months in continuous service.

As operation and maintenance costs vary widely depending on the nature and solids concentration of the sludge being processed, and adjustments to these O&M requirements may have to be made on a case-by-case basis. Conditioning chemical costs are not included in the total annual O&M cost curve (USEPA, 1982a).

Vacuum Filters--Costs for vacuum filter installations are presented in Figure 10-41. The costs include the vacuum filter, conditioning tank, vacuum and filtrate pump assemblies, vacuum receiver, a short belt conveyor for the dewatered sludge, feed sludge piping, lime and ferric chloride storage and feed facilities, electrical controls, and necessary housing for the entire assembly (USEPA, 1982a).

Operation and Maintenance cost are shown in Figures 10-42 and 10-43. Electrical energy curves are presented for bothprocess and building energy. Process energy is for vacuum filer drumdrive, cake discharge roller, vacuum and filtrate pumps, tank agitators, and the dewatered sludge belt conveyor. Process energy requirements were calculated for a sludge solids loading of 17 lb dry 1.7 lb/sq ft/hr. Building sizes are based on conceptual layouts for various total filter areas, and energy requirements are based on 84 kwh/sq ft/yr of building/year (USEPA 1982a).

Labor and maintenance material requirements are based on opeating experience at operating dewatering facilities. Labor requirements are based on 24 hour per day operation, and will have to be adjusted if filters are operated for only one or two shifts per day. Maintenance material costs are for periodic repair and replacement of equipment. Costs are not inlcuded for purchase of the lime or ferric chloride utilized for conditioning, since chemcial requirements are highly variable from sludgeto sludge, and are not generally a function of vacuum filter surface area (USEPA, 1982a).

Table 10-16 shows capital and operating costs for a portable filter press used for dewartering 20,000 gal/yr of 2 percent solids sludge.

FIGURE 10-39. BELT FILTER PRESS—LABOR AND TOTAL ANNUAL OPERATION AND MAINTENANCE COST*

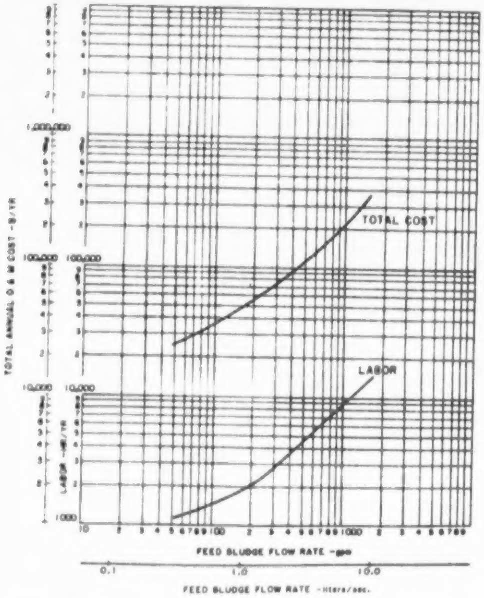
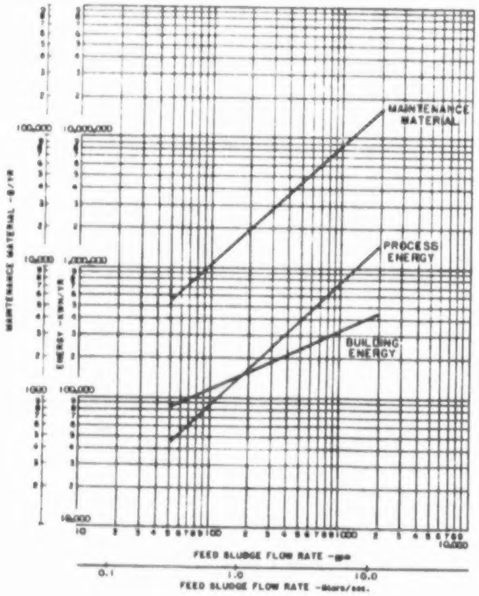


FIGURE 10-40. BELT FILTER PRESS—BUILDING ENERGY, PROCESS ENERGY AND MAINTENANCE MATERIAL REQUIREMENTS*



*Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)

Source: USEPA, 1982a

FIGURE 10-41. CONSTRUCTION COST FOR VACUUM FILTERS*

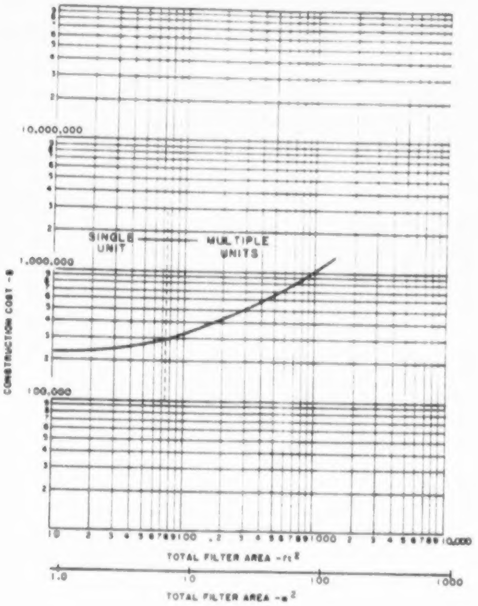
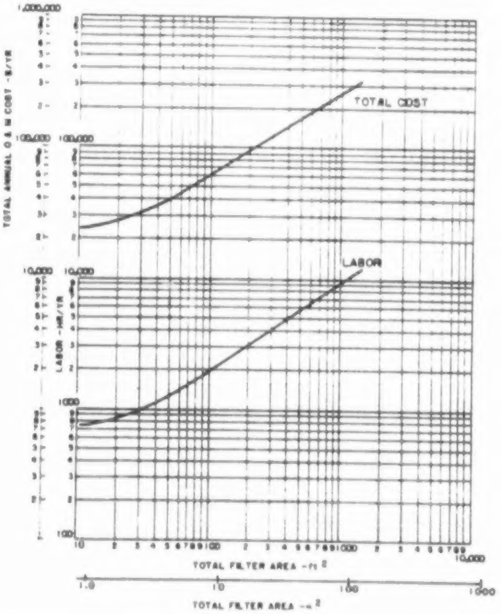


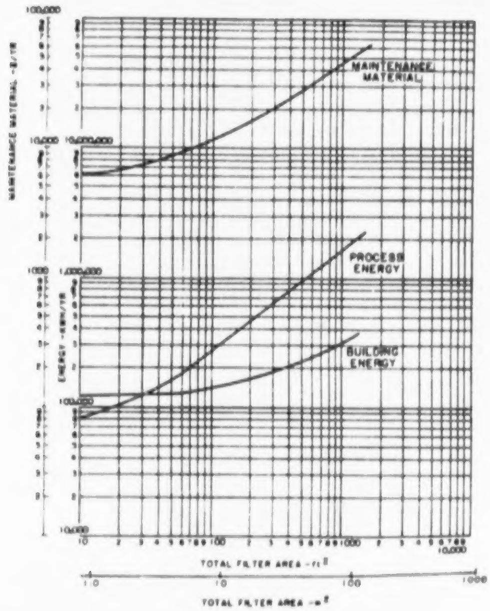
FIGURE 10-42. VACUUM FILTERS—LABOR AND TOTAL ANNUAL OPERATION AND MAINTENANCE COST*



***Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)**

Source: USEPA, 1982a

FIGURE 10-43. VACUUM FILTERS—BUILDING ENERGY, PROCESS ENERGY AND MAINTENANCE MATERIAL REQUIREMENTS



*Costs can be updated to $1985 using ENR Construction Cost Indices for 1982 and 1985 (multiply value shown on this figure by 1.193)

Source: USEPA, 1982a

10.3 Solidification/Stabilization

Solidification and stabilization are terms which are used to describe treatment systems which accomplish one or more of the following objectives (USEPA, 1982b):

- Improve waste handling or other physical characteristics of the waste
- Decrease the surface area across which transfer or loss of contained pollutants can occur
- Limit the solubility or toxicity of hazardous waste constituents.

Solidification is used to describe processes where these results are obtained primarily, but not exclusively, by production of a monolithic block of waste with high structural integrity. The contaminants do not necessarily interact chemically with the solidification reagents, but are mechanically

TABLE 10-16. 1985 CAPITAL AND OPERATING COSTS FOR PORTABLE FILTER PRESS*

The recurring annual costs for dewatering are as follows:

Total labor time to transport, hook-up, operate and wash the press = 40 hrs/month.	=	$ 5,990/yr
Filter cloth cost	=	$ 1,040/yr
Disposal cost	=	$ 6,032/yr
TOTAL		$13,062/yr

The capital cost required is as follows:

24" SPERRY HHC (or equal filter press (delivered) * polypropylene construction * 40" W x 102" L x 62" H * 4,000 lbs. (dry)		$15,600
Trailer and mounting (including a sludge collection pan)	=	$ 2,080
Trailer mounted filtrate return pump	=	$ 2,080
Miscellaneous hoses and filterings	=	$ 1,040
TOTAL		$20,800

*Costs updated to $1985 using 1983 and 1985 ENR construction cost indices.

Source: Moore, Gardner and Assoc., Inc. 1983 locked within the solidified matrix. Contaminant loss is minimized by reducing the surface area. Stabilization methods usually involve the addition of materials which limit the solubility or mobility of waste constituents even though the physical handling characteristics of the waste may not be improved (USEPA, 1982b; Cullinane and Jones, 1985). Methods involving combinations of solidification and stabilization techniques are often used.

Solidification/stabilization methods can be categorized as follows:

- Cement solidification
- Silicate-based processes
- Sorbent materials
- Thermoplastic techniques
- Surface encapsulation
- Organic polymer processes
- Vitrification.

Detailed discussions of solidification/stabilization methods can be found in Guide to the Disposal of Chemically Stabilized and Solidified Waste and Technical Handbook for Solidification/Stabilization of Hazardous Waste (Cullinane and Jones, 1985).

These documents should be consulted for detailed information on these processes. However, it should be noted that the state-of-the-art of solidification/stabilization methods is advancing rapidly. Many manufacturers are marketing processes which involve the use of various combinations of alkaline earth materials (e.g., lime, cement kiln dust, silicaceous materials, cement) often together with organic polymers and proprietary chemicals.

10.3.1 Cement-Based Solidification

10.3.1.1 General Description

This method involves mixing the wastes directly with Portland cement, a very common construction material. The waste is incorporated into the rigid matrix of the hardened concrete. Most solidification is done with Type I Portland cement, but Types II and V can be used for sulfate or sulfite wastes. This method physically or chemically solidifies the wastes, depending upon waste characteristics (USEPA, 1982b). The end product may be a standing monolithic solid or may have a crumbly, soil-like consistency, depending upon the amount of cement added.

10.3.1.2 Applications/Limitations

Most hazardous wastes slurried in water can be mixed directly with cement and the suspended solids will be incorporated into the rigid matrix. Although cement can physically incorporate a broad range of waste types, most wastes will not be chemially bound and are subject to leaching.

Cement solidification is most suitable for immobilizing metals because at the pH of the cement mixture, most multivalent cations are converted into insoluble hydroxides or carbonates. However, metal hydroxides and carbonates are insoluble only over a narrow pH range and are subject to solubilization and leaching in the presence of even midly acidic leaching solutions (e.g., rain). Portland cement alone is also not effective in immobilizing organics.

The end product of cement solidification will not be acceptable for disposal without secondary containment regardless of whether the wastes are organic or inorganic in nature. Another major disadvantage is that cement-based solidification results in wastes that are twice the weight and volume of the original material thereby increasing transportation and disposal costs (USEPA, 1982b). Because of these limitations, Portland cement is generally used only as setting agent in other solidification processes particularly silicate-based processes described in Section 10.3.2.

Another problem with cement solidification is that certain wastes can cause problems with the set, cure, and permanence of the cement waste solid unless the wastes are pretreated. Some of these incompatible wastes are (USEPA, 1982b):

- Sodium salts of arsenate, borate, phosphate, iodate, and sulfide
- Salts of magnesium, tin, zinc, copper, and lead
- Organic matter
- Some silts and clays
- Coal or lignite.

Major advantages to the use of cement include its low cost, and the use of readily available mixing equipment.

10.3.1.3 Implementation Considerations

Since cement is primarily used as a setting agent in other solidification processes, Sections (10.3.2 and 10.3.5) should be consulted for information related to implementation.

10.3.1.4 Costs

Cement costs range from $60 to $90 per ton at the mill. However, capital expenditure and transportation will vary widely depending on the site and the waste (see Section 10.3.2). Cost information for specific wastes should be obtained from vendors. Vendors include: Atcor Washington, Inc., Park Mall, Peeksville, New York; and Chemfix, Inc., Kenner, Louisiana.

10.3.2 Silicate-Based Processes

10.3.2.1 General Description

Silicate based processes refer to a very broad range of solidification/stabilization methods which use a siliceous material together with lime, cement, gypsum, and other suitable setting agents. Extensive research is currently underway on the use of siliceous compounds in solidification. Many of the available processes use proprietary additivies and claim to stabilize a broad range of compounds from divalent metals to organic solvents. The basic reaction is between the silicate material and polyvalent metal ions. The silicate material which is added in the waste may be fly-ash, blast furnace slag or other readily available pozzolanic materials. Soluble silicates such as sodium silicate or potassium silicate are also used. The polyvalent metal ions which act as initiators of silicate precipitation and/or gelation come either from the waste solution, an added setting agent, or both. The setting agent should have low solubility, and a large reserve capacity of metallic ions so that it controls the reaction rate. Portland cement and lime are most commonly used because of their good availability. However, gypsum, calcium carbonate, and other compounds containing aluminum, iron, magnesium, etc. are also suitable setting agents. The solid which is formed in these processes varies from a moist, clay-like material to a hard-dry solid similar in appearance to concrete (Granlund and Hayes, undated).

Some of the additives used in silicate based processes include (Cullinane and Jones, 1985):

- Selected clays to absorb liquid and bind specific anions or cations
- Emulsifiers and surfactants which allow the incorporation of immiscible organic liquids
- Proprietary absorbents that selectively bind specific wastes. These materials may include carbon, zeolite materials and cellulosic sorbents.

There are a number of silicate-based processes which are currently available or in the research stages. Manufacturers' claim differ significantly in terms of the capabilities of these processes for stabilizing different waste constituents.

The Chemfix process uses soluble silicates with cement as the setting agent. Research data shows that the process can stabilize sludges containing high concentrations of heavy metals even under very acidic conditions (Spencer, Reifsnyder, and Falcone, 1982).

The Envirosafe I process uses fly ash as the source of silicates and lime as the alkaline earth material. This method has been shown to stabilize oil bearing sludge (49% oil and grease) and neutralize inorganic metal sludge. Success was demonstrated by use of compressive strength tests (using ASTM methods) and leaching tests (Smith and Zenobia, 1982).

The DCM cement shale silicate process is a proprietary process formulated by Delaware Custom Material, Inc., State College, PA. It involves use of cement, an emulsifier for oily wastes, and sodium silicate. Testing by Brookhaven National Laboratories showed that the process could stabilize oily wastes with up to a 30 percent volumetric loading (Clark, Colombo, and Neilson, 1982). Manufacturers claim that the process can be used to solidify wastes containing acids, organic solvents and oils (Hayes and Granlund, undated).

PQ Corporation of Lafayette Hill, Pennsylvania, has done extensive research on the use of silicates. Their research describes successful stabilization of a mixed heavy metal/organic sludge; a waste containing high levels of organics and petroleum by-products; and a waste containing organic solvents using modifications of the process which involves the use of sodium silicates (Spencer, Reifsnyder, and Falcone, 1982).

10.3.2.2 Applications/Limitations

There is considerable research data to suggest that silicates used together with lime, cement or other setting agents can stabilize a wide range of materials including metals, waste oil and solvents. However, the feasibility of using silicates for any application must be determined on a site-specific basis particularly in view of the large number of additives and different sources of silicates which may be used. Soluble silicates such as sodium and potassium silicate are generally more effective than fly ash, blast furnace slag, etc.

There is some data to suggest that lime-fly ash materials are less durable and stable to leaching that cement fly ash materials (Cullinane and Jones, 1985).

Common problems with lime-fly ash and cement-fly ash materials relate to interference in cementitious reactions that prevent bonding of materials.

Materials such as sodium borate, calcium sulfate, potassium dichromate and carbohydrates can interfere with the formation of bonds between calcium silicate and aluminum hydrates. Oil and grease can also interfere with bonding by coating waste particles (Cullinane and Jones, 1985). However several types of oily sludges have been stabilized with silicate based processes.

One of the major limitations with silicate based processes is that a large amount of water which is not chemically bound will remain in the solid after solidification. In open air, the liquid will leach until it comes to some equilibrium moisture content with the surrounding soil. Because of this water loss, the solidified product is likely to require secondary containment.

Silicate-based processes can employ a wide range of materials, from those which are cheap and readily available to highly specialized and costly additives.

The services of a qualified firm are generally needed to determine the most appropriate formulation for a specific waste type.

10.3.2.3 Implementation Considerations

Commercial cement mixing and handling equipment can generally be used for silicate-based processes. Equipment requirements include chemical storage hoppers, weight or volume-based chemical feed equipment, mixing equipment and waste handling equipment. Ribbon blenders and single and double shaft mixers can be used for mixing. A number of mobile, trailer mounted systems are available.

Silicate-based solidification can also be accomplished on a batch basis in drums. Equipment requirements include on-site chemical storage system, chemical batching system, mixing system, and drum handling system. One company has developed a solidification kit for processing wastes in a drum. The kit consists of a drum containing a disposable mixer blade with the shaft held by bearings welded to the inside of the lid and the bottom of the drum. The upper end of the shaft is accessible through a bung in the lid for turning with an external motor. The cement can be added to the drum before it is capped. The liquid waste and silicate are added through bungs in the lid. An air driven motor is clamped to the drum lid to turn the mixer (Granlund and Hayes, undated).

Solidification can also be accomplished in-situ using a lagoon or mixing pit. This would involve the use of common construction machinery such as a backhor or pull shovel to mix the waste and reagents. However, the ability of in-situ solidification to prevent leaching of contaminants would need to be demonstrated on a case-by-case basis.

10.3.2.4 Costs

Table 10-17 provides estimated costs for silicate cement solidification using three different mixing methods: in-drum mixing, in-situ mixing and a mobile cement mixing system. In all cases it was assumed that 500,000 gallons (2,850 tons) of wastes were solidified with 30 percent portland cement and 2 percent sodium silicate. On-site disposal was assumed. These costs are intended mainly to show the relative cost of various mixing methods and the proportion of total cost for reagents, equipment and labor. It should be emphasized that actual costs are highly waste-and site-specific and that specific site and/or waste characteristics could change these cost estimates by several fold.

In-drum mixing is by far the most expensive alternative and requires the greatest amount of labor and production time. Because of the high cost, in-drum mixing is limited to sites have highly toxic or incompatible wastes in drums (Cullinane and Jones, 1985).

The cost of in-situ mixing and mobile treatment are much more comparable. All are quite sensitive to reagent cost since it typically makes up from 40 to 65% of the total cost. The in-situ technique is the fastest and most economical of the bulk methods because the wastes typically only have to be handled once, or not at all if they are to be left in-place. Labor and equipment each make up less than 5% of the total treatment cost. However, in-situ mixing is the least reliable because of difficulties in accurate reagent measurement and in getting uniform and/or complete mixing of wastes and treatment reagents. Mobile mixing plants, although giving excellent mixing results and reasonably good production rates, require that both the treated and untreated product be handled, thereby increasing the costs above those for in-situ treatment (Cullinane and Jones, 1985).

10.3.3 Sorbents

10.3.3.1 General Description

Sorbents include a variety of natural and synthetic solid materials which are used to eliminate free liquid and improve the handling characteristics of wastes. Commonly used natural sorbent materials include flyash, kiln dust, vermiculite, and bentonite. Synthetic sorbent materials include activated carbon which sorbs dissolved organics, Hazorb (product of Dow Chemical) which sorbs water and organics and Locksorb (product of Radecca Corp.) which is reportedly effective for all emulsions (Cullinane and Jones, 1985).

TABLE 10-17. SUMMARY COMPARISON OF RELATIVE 1985 COST OF STABILIZATION/SOLIDIFICATION ALTERNATIVES[1]

Parameter	In-drum[2]	In-situ[3]	Plant Mixing[4]	
			Pumpable	Unpumpable
Metering and mixing efficiency	Good	Fair	Excellent	Excellent
Processing days required	374	4	10	14
Cost/ton				
Reagent	$ 24.46 (9%)*	$21.27 (63%)	$21.27 (53%)	$21.27 (42%)
Labor and per diem	61.09 (23%)	1.41 (4%)	3.97 (10%)	7.19 (14%)
Equipment rental	44.43 (17%)	1.43 (4%)	4.07 (10%)	7.82 (16%)
Used drums @ $11/drum	57.69 (21%)	-	-	-
Mobilization-demobilization	18.76 (7%)	1.64 (5%)	1.48 (4%)	2.34 (5%)
Cost of treatment process	$206.38	$25.75	$30.79	$38.62
Profit and overhead (30%)	61.91 (23%)	6.73 (23%)	9.29 (23%)	11.59 (23%)
TOTAL COST/TON	$248.29	$33.48	$40.03	$50.21

*% of total cost/ton for that alternative.

[1] Costs updated from 1983 costs using 1985 ENR Index.

[2] Assumed 49 gallons of untreated waste per drum and an average processing rate of 4.5 drums per hour.

[3] Assumed wastes would be mixed by backhoe with a lagoon and left there. Remedial Action is located 200 miles from its nearest equipment.

[4] Assumed pumpable sludge had a daily throughput of 250 yd^3 and the unpumpable sludge a throughput of 180 yd^3/day. Remedial Action is assumed to be located 200 miles from the nearest equipment.

Source: Cullinane and Jones, 1985

10.3.3.2 Applications/Limitations

Sorbents are widely used to remove free liquid and improve waste handling. Some sorbents have been used to limit the escape of volatile organic compounds. They may also be useful in waste containment when they modify the chemical environment and maintain the pH and redox potential to limit the solubility of wastes (Cullinane and Jones, 1985). Although sorbents prevent drainage of free water, they do not necessarily prevent leaching of waste constituents and secondary containment is generally required.

10.3.3.3 Implementation Considerations

The quantity of sorbent material necessary for removing free liquid varies widely depending on the nature of the liquid phase, the solids content of the wate, the moisture level in the sorbent, and the availability of any chemical reactions that take up liquids during reaction. It is generally necessary to determine the quantity of sorbent needed on a case-specific basis.

Equipment requirements for addition and mixing of sorbents are simple. Any of the mixing methods described in Section 10.3.2.3 can be used.

10.3.4 Thermoplastic Solidification

10.3.4.1 General Description

Thermoplastic solidification involves sealing wastes in a matrix such as asphalt bitumen, paraffin, or polyethylene. The waste is dried, heated, and dispensed through a heated plastic matrix. The mixture is then cooled to form a rigid but deformable solid. Bitumen solidification is the most widely used of the thermoplastic techniques.

10.3.4.2 Applications/Limitation

Thermoplastic solidification involving the use of an asphalt binder is most suitable for heavy metal or electroplating wastes. Relative to the cement solidification, the increase in volume is significantly less and the rate of leaching significantly lower. Also, thermoplastics are little affected by either water or microbial attack.

There are a number of waste types which are incompatible with thermoplastic solidification. Oxidizers such as perchlorates or nitrates can react with many of the solidification materials to cause an explosion. Some solvents and greases can cause asphalt materials to soften and never become rigid. Xylene and toluene diffuse quite rapidly through asphalt. Salts that partially dehydrate at elevated temperatures can be a problem. Sodium sulfate hydrate, for example, will loose some water during asphalt incorpoation and if the waste asphalt mix containing the partially dehydrated salt is soaked in water, the mass will swell and crack due to rehydration. This can be avoided by eliminating easily dehydrated salts or coating the outside of the waste/asphalt mass with pure asphalt. Chelating and complelxing agents (cyanides and ammonium) can cause problems with containment of heavy metals (Cullinane and Jones, 1985).

High equipment and energy costs are principal disadvantages of thermoplastic solidification. Another problem is that the plasticity of the matrix-waste mixture generally require that containers be provided for transportation and disposal of materials which greatly increases the cost.

Certain wastes, such as tetraborates, and iron and aluminum salts can cause premature solidification and plug up the mixing machinery (USEPA, 1982b).

10.3.4.3 Implementation Considerations

Thermoplastic solidification requires specialty equipment and highly trained operators to heat and mix the wastes and solidifier. The common range of operating temperatures is 130° to 230°C. The energy intensity of the operation is increased by the requirement that the wastes be thoroughly dried before solidification.

10.3.4.4 Costs

Cost data for thermoplastic solidification outside of the nuclear industry is not readily available. Wernen and Pfleudern Corporation has developed an asphalt binder based process called the Volume Reduction and Solidification System; solidification costs for non-radioactive materials are estimated at $20 to $70 per ton. This cost includes secondary containment but not final transport and disposal (Doyle, 1980).

10.3.5 Surface Microencapsulation

10.3.5.1 General Description

Surface encapsulation describes those methods which physically microencapsulate wastes by sealing them in an organic binder or resin.

Surface encapsulation can be accomplished using a variety of approaches. Three methods which have been the subject of considerable research are described briefly below:

One process developed by Environmental Protection Polymers involves the use of 1,2-polybutadiene and polyethylene (PE) to produce a microencapsulated waste block onto which a high density polyethylene (HDPE) jacked is fused. The 1,2-polybutadiene is mixed with particulated waste which yields, after solvent evaporation, free flowing dry resin-coated particulates. The resulting polymers are resistent to oxidative and hydrolytic degradation and to permeation by water. The next step involves formation of a block of the polybutadiene/waste mixture. Powdered, high density PE is grafted chemically onto the polymer backbone to provide a final matrix with ductile qualities. Various combinations of the two resins (polybutadiene and PE) permit tailoring of the matrix's mechanical properties without reduction of system stability when exposed to severe chemical stress. In the final step, a 1/4 inch thick HDPE jacket is mechanically and chemically locked to the surface of the microencapsulated waste (Lubowitz and Wiles, 1981).

Another encapsulation method developed by Environmental Protection Polymers involves a much simpler approach. Contaminated soils or sludges are loaded into a high density polyethylene overpack. A portable welding apparatus developed by Environmental Protection Polymers is then used to spin weld a lid onto the container thereby forming a seam free encapsulate.

A third surface encapsulation method involves use of an organic binder to seal a cement-solidified mass. United States Gypsum Company manufacturers a product called Envirostone Cement which is a special blend of high-grade polymer modified-gypsum cement. Emulsifiers and ion exchange resins may be added along with the gypsum cement which hydrates to form a freestanding mass. A proprietary organic binder is used to seal the solified mass (United States Gypsum Co., 1982). The process can be used to stabilize both organic and inorganc wastes. It has been shown to effectively immobilize waste oil present at concentration as high as 36 volume percent (Clark, Colombo, and Neilson, 1982). The volume of waste is smaller than that obtained with cement solidification alone.

10.3.5.2 Applications/Limitations

The major advantage of encapsulation processes so far as research shows is that the waste material is completely isolated from leaching solutions.

These methods can be used for both organic and inorganic waste constituents. However, each of the available encapsulation processes are quite unique and the feasibility must be determined on a case-specific basis.

Other advantages associated with hazardous waste encapsulation include (USEPA, 1982b):

- The cubic and cylindrical encapsulates allow for efficient space utilization during transport, storage, and disposal
- The hazard of accidental spills during transport is eliminated
- Materials used for encapsulation are commercially available, very stable chemically, nonbiodegradable, mechanically tough, and flexible
- Encapsulated waste materials can withstand the mechanical and chemical stresses of a wide range of disposal schemes (landfill, disposal in salt formations, ocean disposal).

The major disadvantages associated with encapsulation techniques include:

- Binding resins required for agglomeration/encapsulation (high density polyethylene; polybutadiene) are relatively expensive
- The processes are energy intensive and relatively costly
- Skilled labor is required to operate molding and fusing equipment.

10.3.5.3 Costs

Environmental Protection Polymers has estimated that the cost of the polybutadiene/HDPE microencapsulation method will be approximately $90/ton. Encapsulation in the seam-free HDPE overpack is approximately $50 to $70 for a 80 gallon drum load (Lubowitz, H., Environmental Protection Polymers, personal communication October 13 and 14, 1983).

10.3.6 Vitrification

10.3.6.1 General Description

Vitrification of wastes involves combining the wastes with molten glass at a temperature of 1,350°C or greater. However, the encapsulation might be done at temperatures significantly below 1,350°C (a simple glass polymer such as boric acid can be poured at 850°C). This melt is then cooled into a stable, noncrystalline solid (USEPA, 1982b).

10.3.6.2 Applications/Limitations

This process is quite costly and so has been restricted to radioactive or very highly toxic wastes. To be considered for vitrification, the wastes should be either stable or totally destroyed at the process temperature.

Of all the common solidification methods, vitrification offers the greatest degree of containment. Most resultant solids have an extremely low leach rate. Some glasses, such as borate-based glasses, have high leach rates and exhibit some water solubility. The high energy demand and requirements for specialized equipment and trained personnel greatly limit the use of this method.

10.3.6.3 Implementation Considerations

Glassification of wastes is an extremely energy intensive operation and requires sophisticated machinery and high trained personnel.

10.3.6.4 Cost

No cost information was available for glassification.

10.3.7 Technology Selection/Evaluation

Evaluation of the technical feasibility and effectiveness of stabilization/solidification methods must be determined on a case-by-case basis. Commercial firms specializing in these processes should be consulted whenever solidification/stabilization is being considered. Samples of the solidified product will need to be subjected to extensive leaching tests unless a reliable, effective means of secondary containment is to be used. It should be noted that secondary containment is recommended with most of the previously described methods (except microencapsulation and glassification for some waste types). Similarly, where the end product is intended to be a monolithic block, samples must be subjected to compressive strength tests.

Solidification/stabilization methods run the gamut from those which use simple, safe, readily available equipment (cement and most silicate-based processes) to those which require highly sophisticated, costly, and specialized equipment (e.g., glassification and thermoplastic techniques). Use of these high technology processes should be limited to wastes which cannot be treated cost-effectively using any other methods. Regardless of the simplicity of some of the equipment, professionals trained in these processes should be consulted since formulations including proprietary additives are very waste specific.

10.4 Gaseous Waste Treatment

Gaseous wastes may be present at hazardous waste sites as a result of bulk gas disposal in containers, volatilization of organic liquids, byproducts of waste decomposition, or byproducts of treatment or other on-site processes. Once captured and collected, gaseous wastes can be treated thermally to render contaminants less hazardous or treated physically or chemically to remove and concentrate contaminants. The following three categories of gaseous waste treatment are described in this section:

- Incineration
- Flaring
- Adsorption.

10.4.1 Flaring

10.4.1.1 General Description

Combustion is a chemical reaction that thermally oxidizes a substance into products that generally include ash, gases, water vapor, and heat. Flaring is a special category of combustion where wastes are exposed to an open flame and no special features are employed to control temperatures or time of combustion. Supplementary fuels may be needed to sustain continuous combustion.

10.4.1.2 Applications/Limitations

Flares are commonly used in the oil and gas industry to dispose of waste gases and fumes at refineries; at sewage treatment plants to dispose of digestor gas; and at sanitary landfills to dispose of landfill gas. Although flares provide sufficient destruction of contaminants for conventional applications, destruction removal efficiencies (DREs) required by current environmental regulations for thermal destruction of hazardous wastes are generally too stringent to be met by flaring. Exceptions may be gaseous waste streams consisting of relatively simple hydrocarbons (emissions from fuel tanks, landfill methane gas, etc.).

Supplemental fuel is required to sustain a flame with gases of low heating value. Gases with heating values as low as the low hundreds of Btu's per cubic foot can sustain a flame (natural gas has a heating value of approximatley 1,000 Btu's per cubic foot).

Flame sensors, pilot flames, automatic sparkers, and alarms are often used to sense loss of flame, attempt reignition, and alert operators to system performance problems. Shields can be placed around floors to serve as windbreaks for containing and preventing "blowing out" of the flame.

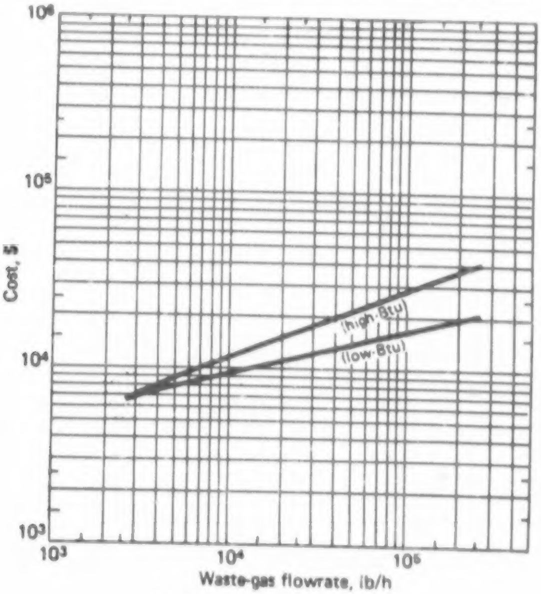
The flow rate dictates the diameter and height of the flare and the number of flares to be employed. The flare must be designed such that the flame is largely contained within the body of the flare for safety reasons and to allow adequate mixing of gas and air. The oxygen content of the gas influences the air/gas ratio that is sought in the combustion area of the flare.

10.4.1.3 Costs

Purchase costs of flares depend primarily on the waste-gas flowrate, and secondarily on design and elevation. Costs in dollars for flowrates between 1,000 and 100,000 lb/h are given in Figure 10-44. Costs include ladders, platforms, knockout drums with seals, and stacks high enough to ensure grade-level radiation no greater than 1,500 Btu/(h)(ft^2). Costs described in Figure 10-44 refers to self-supporting type elevated flares (approximately 40 feet high). Costs for elevated flares supported by guyed wires (nominally 100 feet tall) range from 30 percent higher (than the self-supporting type) at 250,000-lb/h flowrates, to 80 percent higher at 2,500-lb/h flowrates.

Operating costs for flares are high because of the substantial quantity of natural gas and steam (in the smokeless type) consumed. If the waste-gas must be driven, fan power costs for overcoming pressure drops may also be

FIGURE 10-44. PURCHASE COSTS OF ELEVATED FLARES



(high Btu—ethylene)
(low Btu—60 Btu/ft^3)

Source: Vatavuk and Neveril, 1983

high. This pressure drop depends on the size of the flare, knockout drum, puiping and stack height, with the maximum allowable pressure drop being approximately 60 in. H_2O.

Elevated flares require supplemental fuel, (in addition to gas for pilots and purging) when a low-Btu gas is being burned. The supplemental fuel (natural gas) required is plotted against waste-gas flowrate in Figure 10-45. This graph is based on an 880-h/yr operating factor, for gas flowrates in the range of 1,000 to 250,000 lb/h.

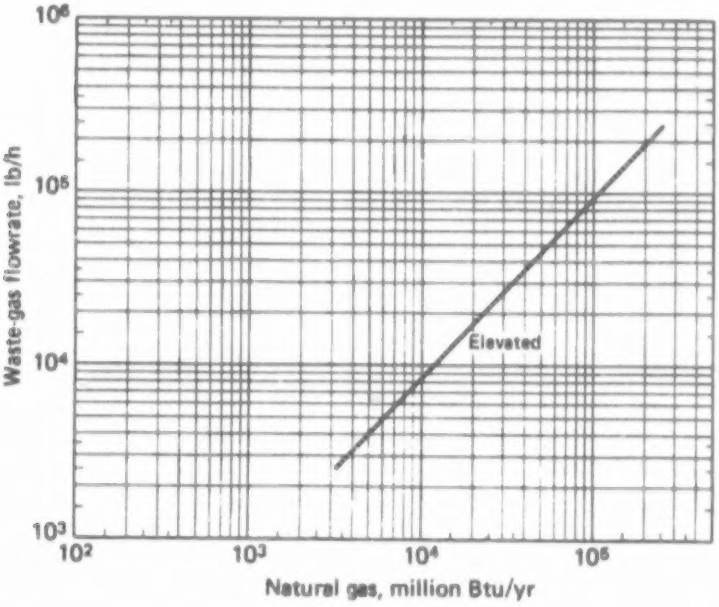
Steam consumption for smokeless flares (or others requiring steam injection) can be estimated at 0.6 lb/lb of waste-gas.

10.4.1.4 Technology Selection/Evaluation

Flaring systems, by virtue of their relative lack of controllability, are generally considered to perform inconsistently. They are relatively simple to both fabricate and install. Conventional steel plate, pipe, and welding are employed in fabrication.

When properly designed and operated, flares pose no unusual safety impacts to operators or others. The presence of a visible flame is sometimes considered by the public to be a nuisance.

FIGURE 10-45. NATURAL GAS REQUIREMENTS FOR ELEVATED FLARES



Source: Vatavuk and Neveril, 1983

Properly designed flaring systems operate automatically. The most basic design of flaring requires monical ignition. Pilot flames, sensors, and igniters may require regular maintenance. Monitoring of air quality local to the flame is advisable to ensure that adequate treatment is being afforded.

10.4.2 Adsorption

10.4.2.1 General Description

Adsorption is the adherence of one substance to the surface of another by physical and chemical processes. Treatment of wastestreams by adsorption is essentially a process of transferring and concentrating contaminants (the adsorbate) from one medium (liquid or gas) to another (the adsorbent). The most commonly used adsorbent is activated carbon; generally, the granular form (GAC) is used to treat gaseous wastes. Other adsorbents are specially manufactured resins.

Activated carbon is a highly porous material. Adsorption takes place on the walls of the pores because of an imbalance of forces on the atoms of the walls. The adsorption of molecules onto the wall surfaces serves to balance the forces (Calgon Corp., undated). Adsorption onto resins occurs in a similar fashion.

Adsorption gas treatment systems consist of containerized beds of adsorbent. Large and highly permeable void spaces between relatively large GAC particles or pellets (nominal size of several millimeters) allow the contaminated medium to flow through the bed, contacting the particles and allowing adsorption to take place. The treated medium leaves the bed with reduced concentrations of adsorbate until the adsorbent has reached capacity. Once adsorbents have reached capacity, little or no further adsorption occurs and some contaminants can be released back into the medium (desorption) and actually increase contaminant concentrations.

Adsorbents at capacity can be disposed of in appropriate landfills, incinerated, or can be regenerated, driving off the adsorbate and allowing reuse of the adsorbent for treatment. GAC is regenerated by heating in a reduced-pressure atmosphere (Calgon Corp, undated). Resins are regenerated by washing with appropriate solvents (Kiang and Metry, 1982). The adsorbate can be recovered and reused (solvents, for example) from the regeneration process.

Multiple bed vessels are often required to allow adequate contact time and/or to optimize the frequency of adsorbent changeover or regeneration. Partial or total redundant capacity is often provided by extra bed vessels to allow continuous operation during changeover or regeneration.

10.4.2.3 Applications/Limitations

Carbon adsorption is generally accepted for use in controlling volatile hydrocarbons; organic-related emissions; certain sulfur-related emissions such as hydrogen-sulfide; mercury; vinyl chloride; most halogenatd organics; and radioactive materials such as iodine, krypton, and xenon. Carbon adsorption can also control oxides of sulfur and nitrogen and carbon monoxide (Calgon Corp., undated).

Resins are capable of removing most organic contaminants from water and are also applicable to removal of organics from gaseous streams. However, resins are not widely used for gaseous waste treatment.

Adsorption is widely used in industry for air pollution and odor control, often in association with solvent recovery and reuse systems. Generally, GAC acts as an accumulator of organic contaminants until the bed is saturated. Hot gases are passed through the bed to desorb the organics which are condensed and recovered or are incinerated (Calgon Corp., undated).

Monitoring of gas flowrate and influent and discharge adsorbate concentrations are needed to determine changeover/regeneration schedules. Automatic monitors and microprocessors may be warranted for highly complex and variable systems. Alarms and/or shut-down controls may also be warranted for complex systems or in sensitive or populated areas.

10.4.3 Technology Selection/Evaluation

Adsorption techniques are well-established for removal of organic compounds and some inorganic compounds from gaseous streams. Adsorption is highly reliable provided that adsorbate and adsorbent are properly matched, sufficient contact time is allowed, and the adsorbent is regenerated or replaced before saturation (and desorption) is reached. Many adsorption systems are prepackaged and can be quickly installed and placed into operation by contractors, suppliers, or manufacturers. Specially designed systems employ off-the-shelf towers, blowers, and other equipment, and require additional installation time.

Operation of properly designed adsorption gas treatment systems is essentially as automatic as the gas delivery system although manual or special automatic adjustments may be warranted for highly variable flows or adsorbate concentrations. Changeover or regeneration of the adsorbent bed must be conductd on a predetermined basis to ensure continuous effective treatment.

10.5 Thermal Destruction of Hazardous Wastes

Thermal destruction is a treatment method which uses high temperature oxidation under controlled conditions to degrade a substance into products that generally include CO_2, H_2O vapor, SO_2, NO_x, HCl gases and ash. The hazardous products of the thermal destruction/incineration such as particulates, SO_2, NO_x, HCl, and products of incomplete combustion require air pollution control equipment to prevent release of undesirable species into the atmosphere. Thermal destruction methods can be used to destroy organic contaminants in liquid, gaseous and solid waste streams.

The most common incineration technologies applicable to hazardous wastes include:

- Liquid injection
- Rotary kiln
- Fluidized bed
- Multiple hearth

The operating principles and general applications of these methods are summarized in Table 10-18. Mobile incinerators, at sea incinerators and coincineration commonly employ these technologies.

Emerging technologies for the thermal destruction of wastes include (Monsanto Research Corp.; 1981, Keitz and Lee, 1983; Lee, 1983; State of California, 1981):

- Molten salt
- Wet air oxidation
- Plasma arch torch
- Circulating bed
- High temperature fluid wall
- Pyrolysis
- Supercritical water
- Advanced electric reactor
- Vertical tube reactor.

TABLE 10-18. SUMMARY OF COMMONLY USED INCINERATION TECHNOLOGIES

Type	Process Principle	Application	Combustion Temperature	Residence Time
Rotary Kilns	Waste is burned in a rotating, refractory cylinder	Any combustible solid, liquid, or gas	1500-3000°F	Seconds for gases to hours for liquids and solids
Single Chamber/ Liquid Injection	Wastes are atomized with high pressure air or steeam and burned in suspension	Liquids and slurries which can be pumped	1300-3000°F	0.1 to 1 second
Multiple Hearth	Wastes descend through several grates to be burned in increasingly hotter combustion zones	Sludges and granulated solid wastes	1400-1800°F	Up to several hours
Fluidized-Bed Incineration	Waste is injected into an agitated bed of heated inert particles. Heat is efficiently transferred to the wastes during combustion	Organic liquids, gases and granular or well processed solids	1400-1600°F	Seconds for gases and liquids, minutes for solids

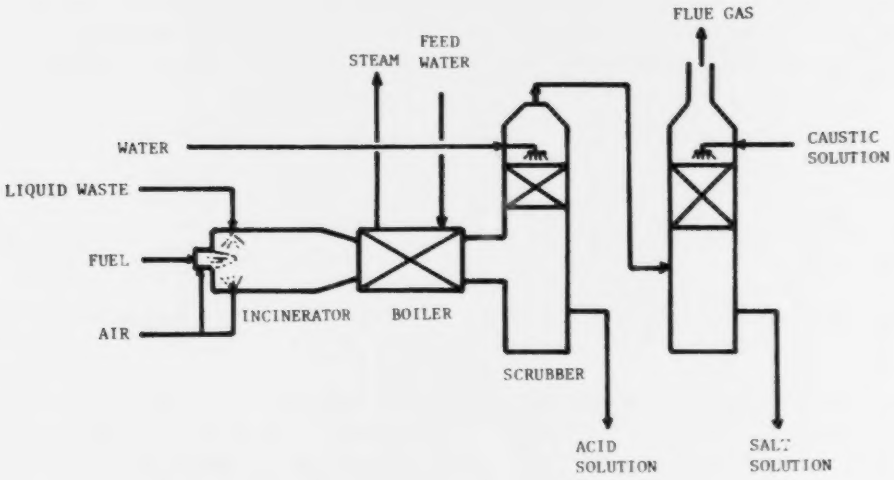
Source: State of California, 1981

10.5.1 Liquid Injection

10.5.1.1 General Description

A liquid incineration system consists of a single or double refractory-lined combustion chamber and a series of atomizing nozzles. Two chamber systems are more common. The primary chamber is usually a burner where combustible liquid and gaseous wastes are introduced. Noncombustible liquid and gaseous wastes are introduced downstream of the burner in the secondary chamber. A schematic diagram of a two-stage system is shown in Figure 10-46. Single chamber incinerators are used for systems handling only combustible wases (Kiang and Metry, 1982).

FIGURE 10-46.
LIQUID INJECTION INCINERATION SYSTEM



Source: Kiang and Metry, 1982

The most popular liquid injection incinerators are horizontally and vertically fired units. A liquid waste has to be converted into a gas before combustion. The liquid is atomized passing through the burner nozzles while entering the combustor. This is necessary to ensure complete evaporation and oxidation. If viscosity precludes atomization, mixing and heating or other means should be applied prior to atomization to reduce waste viscosity.

The operating temperatures vary from 1300 to 3,000°F, with the most common temperature being about 1600°F. Residence times vary from less than 0.5 seconds to 2 seconds (Lee, Keitz, and Vogel, 1982; State of California, 1981). The process usually requires 20 to 60% excess air to ensure complete combustion.

10.5.1.2 Application Limitations

Liquid injection can be used to destroy virtually any pumpable waste or gas. These units have been used in the destruction of PCBs, solvents, still and reactor bottoms, polymer wastes, and pesticides (State of California, 1981). Unlikely candidates for destruction include heavy metal wastes and other wastes high in inorganics. It does not have a need for a continuous ash removal system other than for pollution control (Monsanto Research Corp., 1981).

Liquid incinerators have no moving parts and require the least maintenance of all types of incinerators. The major limitations of liquid injection are its ability to incinerate only wastes which can be atomized in the burner nozzle and the burner's susceptibility to clogging. It also needs a supplemental fuel.

Liquid injection incinerators are highly sensitive to waste composition and flow changes. Therefore, storage and mixing tanks are necessary to ensure a reasonably steady and homogenous waste flow (Kiang and Metry, 1982).

10.5.2 Rotary Kiln

10.5.2.1 General Description

Rotary kilns are capable of handling a wide variety of solid and liquid wastes.

Rotary kiln incinerators are cylindrical, refractory-lined shells. They are fueled by natural gas, oil, or pulverized coal. Most of the heating of the waste is due to heat transfer with the combustion product gases and the walls of the kiln. The basic type of rotary kiln incinerator, illustrated in Figure 10-47, consists of the kiln and an afterburner (Kiang and Metry, 1982).

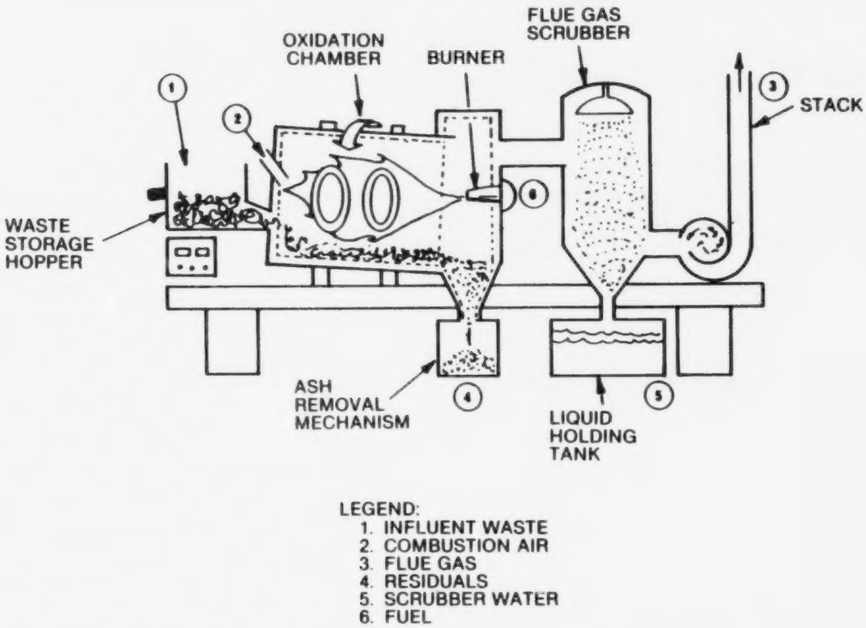
Wastes are injected into the kiln at the higher end and are passed through the combusion zone as the kiln rotates. The rotation creates turbulence and improves combustion. Rotary kilns often employ afterburners to ensure complete combustion. Most rotary kilns are equipped with wet scrubber emission controls.

The residence time and temperature depend upon combustion characteristics of the waste. Residence times can range from a few seconds to an hour or more for bulk solids. Combustion temperature range from 1500 to 3000°F.

10.5.2.2 Application/Limitations

Rotary kilns are capable of burning waste in any physical form. They can incinerate solids and liquids independently or in combination and can accept waste feed without any preparation (Monsanto, 1981). Hazardous wastes which have been treated in rotary kilns include PCBs, tars, obsolete munitions, polyvinyl chloride wastes, and bottoms from solvent reclamation operations (State of California, 1981).

FIGURE 10-47. ROTARY KILN INCINERATOR SCHEMATIC



Source: Ghassami, Yu, and Quinlivan, 1981

Because of their ability to handle waste in any physical forms, and their high incineration efficiency, rotary kilns are the preferred method for treating mixed hazardous solid residues (Lee, Keitz, and Vogel, 1982).

The limitations of rotary kilns include susceptibility to thermal shock, the necessity for very careful maintenance, need for additional air due to leakage, high particulate loadings, relatively low thermal efficiency, and a high capital cost for installation (Monsanto Research Corp., 1981).

10.5.3 Multiple Hearth

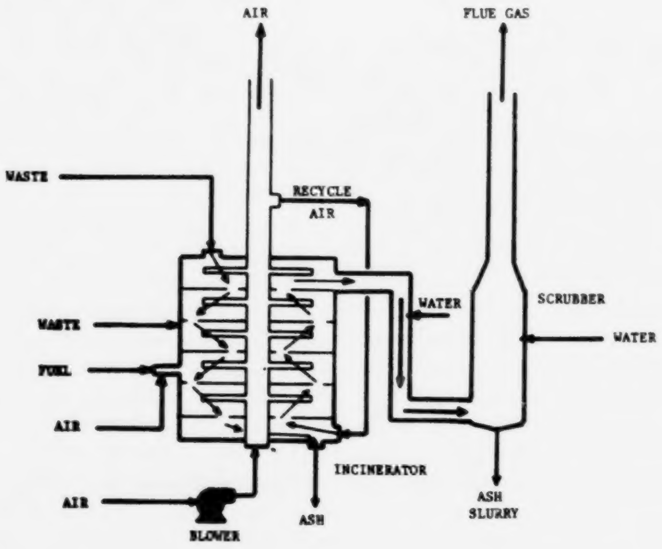
10.5.3.1 General Description

A multiple hearth incinerator consists of a refractory lined steel shell, a rotating central shaft, a series of solid flat hearths, a series of rabble arms with teeth for each hearth, an air blower, waste feeding and ash removal systems, and fuel burners mounted on the walls (Monsanto Research Corp., 1981). Figure 10-48 illustrates the components of the multiple hearth. It can also be equipped with an afterburner, liquid waste burners and side ports for tar injection. Temperature in the burning zone ranges from 1400 to 1800°F and residence time may be very long.

10.5.3.2 Applications/Limitations

The multiple hearth incinerator can be used for the disposal of all forms of combustible industrial waste materials, including sludges, tars, solids, liquid and gases. The incinerator is best suited for hazardous sludge destruction. Solid waste often requires pretreatment such as shredding and

FIGURE 10-48. MULTIPLE HEARTH INCINCERATOR



Source: Kiang and Metry, 1982

sorting. It can treat the same wastes as the rotary kiln provided that pretreatment of solid waste is applied. The principal advantages of multiple hearth incineration include high residence time for sludge and low volatile materials; ability to handle a variety of sludges; ability to evaporate large amounts of water; high fuel efficiency and the utilization of a variety of fuels. The greatest disadvantages of the technology include susceptibility to thermal shock; inability to handle wastes containing ash, which fuses into large rock-like structures, and wastes requiring very high temperatures. Also control of the firing of supplemental fuels is difficult. The multiple hearth incinerator has high maintenance and operating costs (Monsanto Research Corp., 1981; State of California, 1981). The operating cost may be reduced by utilizing liquid or gaseous combustible wastes as secondary fuel (Monsanto Research Corp., 1981).

10.5.4 Fluidized Bed

10.5.4.1 General Description

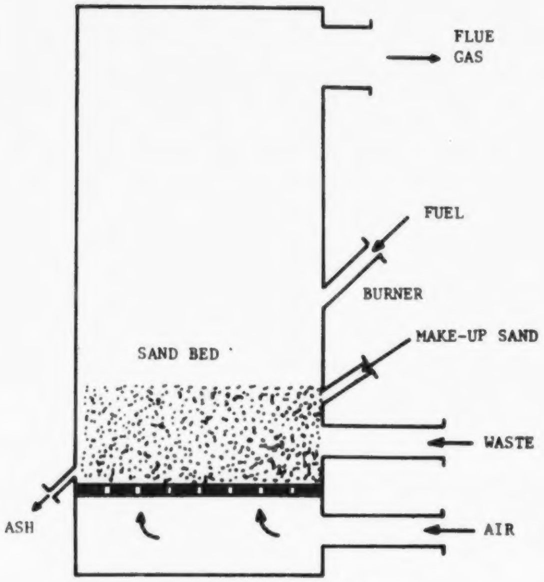
The fluidized bed incinerator illustrated in Figure 10-49 consists of a cylindircal vertical refractory lined vessel containing a bed of inert granular material, usually sand on a perforated metal plate. Combustion air is introduced through a plenum at the bottom of the incinerator and rises vertically fluidizing the bed and maintaining turbulent mixing of bed particles. Waste material is injected into the bed and combustion occurs within the bubbling bed. Heat is transferred from the bed into the injected wastes. Auxiliary fuel is usually injected into the bed. Bed temperatures vary from 1400 to 1600°F. Since the mass of the heated, turbulent bed is much greater than the mass of the waste, heat is rapidly transferred to the waste materials; a residence time of a few seconds for gases and a few minutes for liquids is sufficient for combustion (State of California, 1981).

The residence time is long enough to allow the solid materials to become small and light enough to be carried off as particulates. Suspended fine particulates are usually separated in a cyclone when exhaust gases pass through air pollution control devices before being released into the atmosphere.

10.5.4.2 Applications/Limitations

Fluidized bed incinerators are a relatively new design, presently being applied for liquid, solid and gaseous combustible wastes. The most typical wastes treated in fluidized beds include slurries and sludges. Some wastes require pretreatment prior to entering the reactor. The pretreatment may involve drying, shredding and sorting. The fluidized bed handles the same waste that can be treated in the rotary kiln (Monsanto Research Corp., 1981).

FIGURE 10-49. FLUIDIZED BED INCINERATOR



Source: Kiang and Metry, 1982

Fluidized beds are typically used for the disposal of municipal wastewater treatment plant sludges, oil refinery waste, and pulp and paper mill waste. There is only limited data on the use of fluidized bed for hazardous waste incineration. The technology has been used for pharmaceutical wastes, phenolic wastes, and methyl methacrylate (State of California, 1981).

It is particularly well suited for incineration of high-moisture wastes, sludges and wastes containing large quantities of ash. Because of the low bed temperature, the exhaust gases usually contain low nitrogen oxides (Kiang and Metry, 1982).

The advantages of the fluidized bed incinerator include simple design, minimal NO_x formation, long life of the incinerator, high efficiency, simplicity of operation, and relatively low capital and maintenance costs. It also has the ability to trap some gases in the bed, reducing the need for and the cost of an emission control system. The disadvantages include difficulty in removing residual materials from the bed, a relatively low throughput capacity, the difficulty in handling residues and ash from the bed and the relatively high operating costs (State of California, 1981; Monsanto Research Corp., 1981).

10.5.5 At-Sea Incineration, Mobile Incineration, and Coincineration

At-sea incineration usually utilizes a liquid injection unit mounted on a ship, to destroy hazardous waste far away from populated areas and shipping lanes. No acid gas pollutant removal system is applied. The wastes treated include toxic organochloride compounds, herbicides, and Agent Orange. The basic advantage of at-sea incinertaion is the distance from populated areas and the high efficiency of combustion. The disadvantages are problems with monitoring an at-sea process, the danger of spills and the need to operate on-shore auxiliary facilities.

Mobile incineration has not been widely used yet but the demand for the application of this technique grows with future bans on the landfill disposal of certain wastes. Existing mobile systems include liquid injection and rotary kiln incinerators equipped with secondary combustion chambers and environmental controls. These mobile incinerators are capable of handling a variety of wastes including PCBs, carbon tetrachloride, other hazardous wastes and soils. The primary advantage of the mobile incinerator is its ability to treat on-site and thus eliminate the need for off-site transport of waste. There is currently only limited experience with mobile incinerators. Mobile incinerators must meet all applicable state requirements which typically include air emission permits.

Coincineration is a process of using combustible wastes as supplemental fuels in fossil fuel boilers or any type of incinerator (State of California, 1981; Monsanto Research Corp., 1981). As a result of coincineration, the energy value of the waste is used to produce steam and the original form of the waste is destroyed through combustion. This incineration technique can be implemented in any boiler where the parameters of combustion and feed make it feasible. The principal advantages of coincineration include low capital cost, and no need for the transport of on-site generated wastes. Disadvantages include the possibility of damaging the boiler by some harmful waste and the difficulty in obtaining high efficiency of combustion.

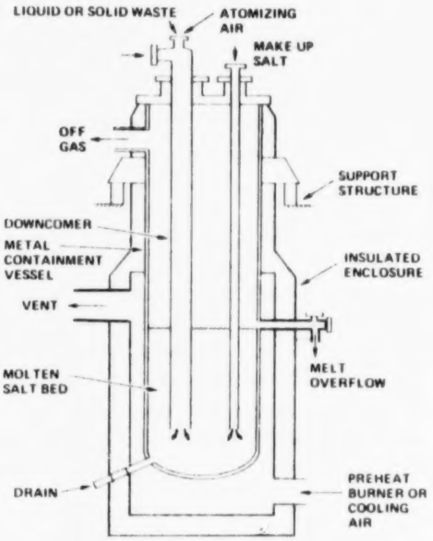
10.5.6 Advanced Incineration Technologies

10.5.6.1 Molten Salt

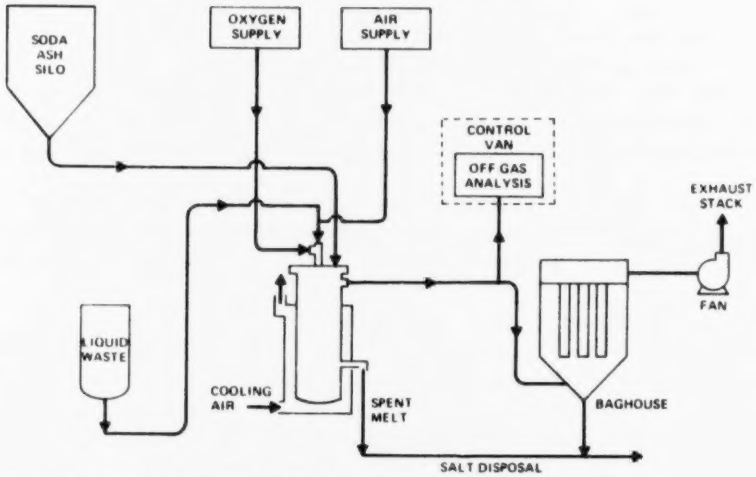
The molten salt incinerator can be used for destruction of hazardous liquids and solids. In this method (illustrated in Figure 10-50) wastes undergo catalytic destruction when they contact hot molten salt maintained at a temperature between 1382 and 1832°F (Ross, 1984; Solsberg, Parent, and Ross, 1985). Hot gases rise through the molten salt bath, pass through a secondary

FIGURE 10-50. MOLTEN SALT INCINERATOR

MOLTEN SALT COMBUSTOR SCHEMATIC



SIMPLIFIED FLOW SCHEMATIC MOLTEN SALT DESTRUCTION



Source: Rockwell International, 1980

reaction zone, and through an off-gas clean-up system before discharging to the atmosphere (Kiang and Metry, 1982). Supplemental fuel may be required when wastes are not sufficiently combustible to maintain temperatures.

Liquid, free-flowing powders, sludges, and shredded solid wastes can be fed directly into the incinerator. The technology has been demonstrated to be highly effective for chlorinated hydrocarbons including PCB, chlorinated solvents, and malathion (Kiang and Metry, 1982). However, the process appears to be sensitive to materials containing high ash content or high chlorine content which must ultimately be removed in the purge system (Ross, 1984).

10.5.6.2 Wet Air Oxidation

Wet air oxidation involves aqueous phase oxidation of dissolved or suspended organic substances at elevated temperatures and pressures. The temperature of the process is relatively low, 350-650°F, and the pressure varies between 300-3000 PSI. Figure 10-51 shows a simplified scheme of the wet air oxidation process.

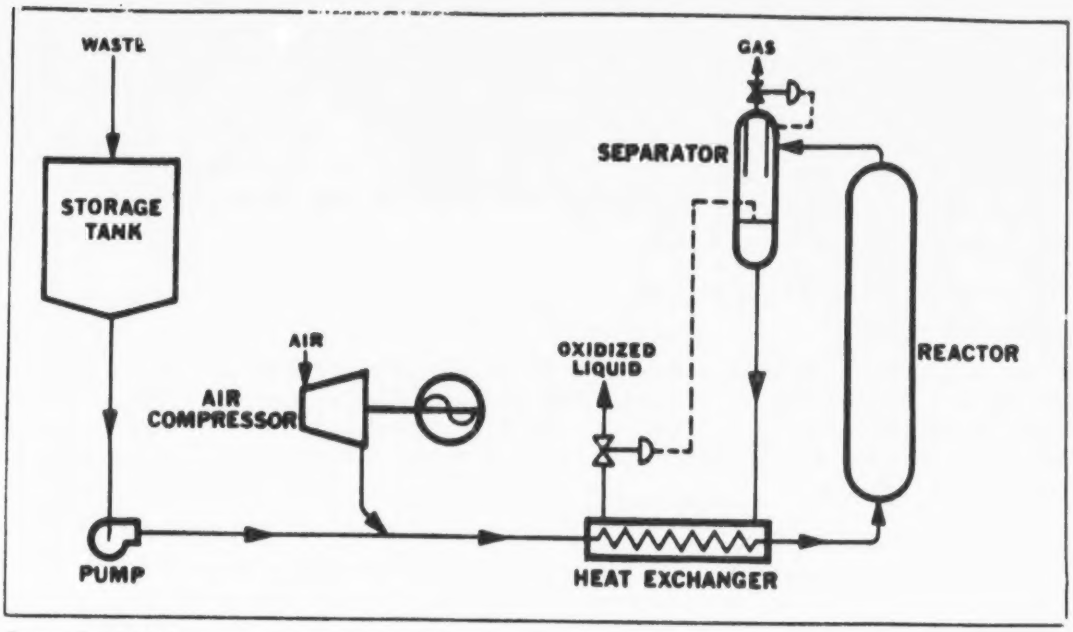
The waste is pumped into the system by the high pressure pump and mixed with air from the air compressor. The mixture passes through a heat exchanger, and then into the reactor where oxygen in the air reacts with organic matter in the waste. The oxidation is accompanied by a temperature rise. The gas and liquid phases are separated after the reactor, and the liquid passes through the heat exchanger, heating the incoming material. The gas and liquid streams are discharged from the system through control valves. The degree of oxidation is primarily a function of reaction temperature and residence time.

WAO is used primarily to treat concentrated waste streams containing organic and oxidizable inorganic wastes. It is generally selected for treating or pretreating a waste stream which has a high COD/BOD_5 ratio and is not readily amenable to biological treatment. It is also selected where it is determined to be more cost-effective than incineration. Waste streams for which WAO is particularly applicable include concentrated streams containing pesticides, herbicides or other complex organics which are not readily biodegradable.

10.5.6.3 Plasma Arc Torch

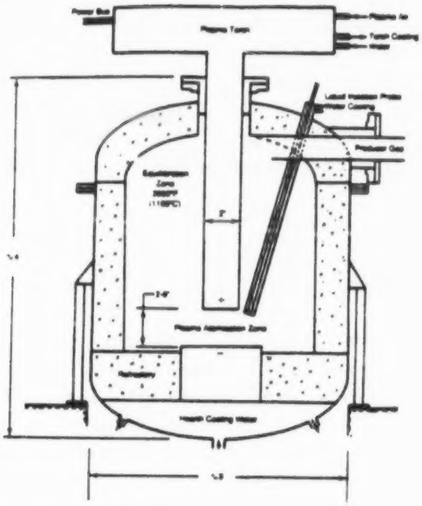
Plasma arc torch may be used to destory either liquid or solid wastes by pyrolyzing them into combustible gases in contact with a gas which has been energized to its plasma state by an electrical discharge (see Figure 10-52). The plasma gas temperature is about 90,000°F. Wastes are atomized, ionized

FIGURE 10-51.
FLOWSHEET OF WET AIR OXIDATION



Source: Pradt, 1976

FIGURE 10-52.
PLASMA REACTION VESSEL SCHEMATIC



Source: Lee, Keitz and Vogel, 1982

and destroyed in contact with the plasma. The advantages of this method are possibilities of using the exiting gas as a fuel (after removal of halogens and other contaminants in a scrubber), the lack of hazardous interim combustion products, high efficiency and the ability to be portable. Costs are not presently available (State of California, 1981).

10.5.6.4 Circulating Bed Combustion (CBC)

Circulating bed combustion is an outgrowth of conventional fluidized bed incineration. However, the fluid bed operates with higher velocities than conventional fluid beds and it recirculates the fluidized material within the system returning it back to the feed section (Ross, 1984). Figure 10-53 illustrates a CBC.

The CBC is suitable for burning solid, liquid, sludge or gaseous waste streams. The advantages of this incinerator are similar to those of a conventional fluidized bed system with lower susceptibility to corrosion of the boiler, a less complicated scrubbing system, close temperature control and dry solid waste recovery.

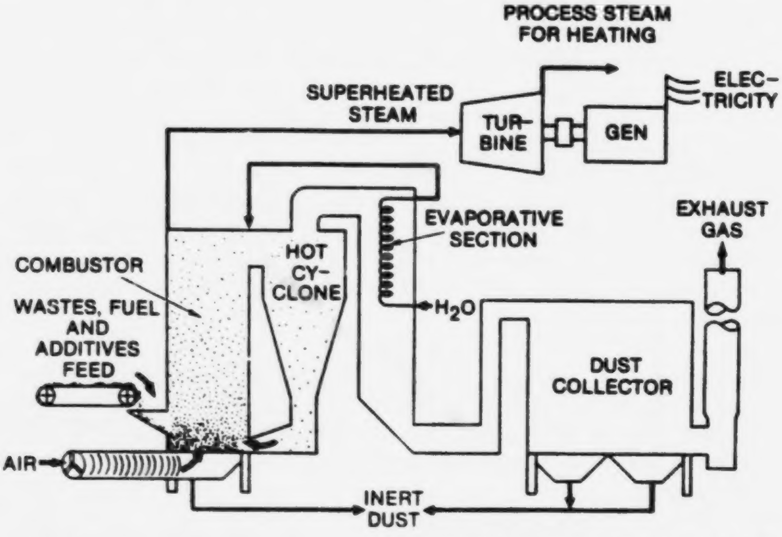
10.5.6.5 High Temperature Fluid Wall (HTFW)

The high temperature fluid wall process quickly reduces organic wastes to their elemental state in a very high temperature process (about 4000°F) (Keitz and Vogel, 1982). The process is carried out in a patented reactor which consists of a tubular core of refractor material capable of emitting radiant energy supplied by large electrodes in the jacket of the vessel. During the process, an inert gas is injected to coat the wall of the reactor and prevent destruction from high temperatures. A cross-section of a typical high-temperature fluid wall reactor is shown in Figure 10-54. HTFW has been used to treat PCB contaminated earth and other wastes. It ensures high destruction efficiency, eliminates the formation of intermediate pyrolysis products but requires some preparation of the feed material and it also incurs high energy costs.

10.5.6.6. Pyrolysis

Pyrolysis is the thermal conversion of organic material into solid, liquid and gaseous components. Pyrolysis takes place in an oxygen-deficient atmosphere at temperatures from 900° to 1600°F. The volatile organics generated in the process are burned in a second stage fume incinerator at temperatures of 1800 to 3000°F. The two-stage process minimizes the volatilization of inorganic components and ensures that inorganics, including heavy metals, form an insoluble solid char residue. The technology may be used for the destruction of materials containing carbon, hydrogen and oxygen. Pyrolysis can not handle wastes with nitrogen, sulfur, sodium contents.

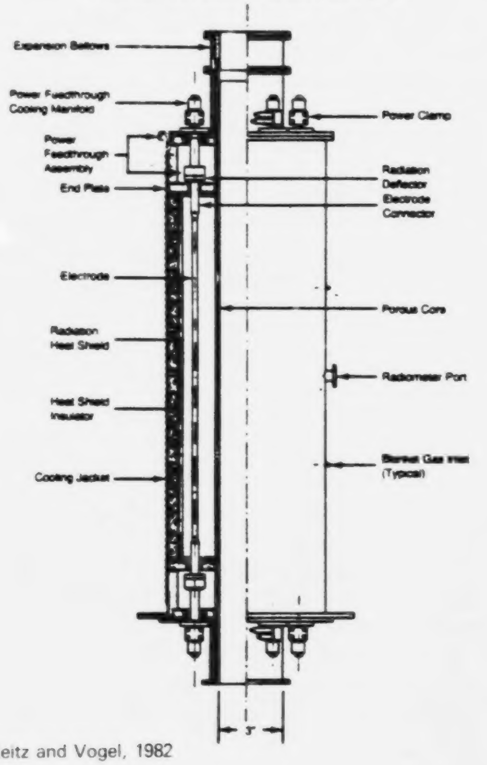
FIGURE 10-53. CIRCULATING BED COMBUSTOR



Source: Ross, 1984

TURNS LOW GRADE FUEL INTO POWER

FIGURE 10-54. CROSS-SECTION OF A TYPICAL HIGH-TEMPERATURE FLUID-WALL REACTOR [E]



Source: Lee, Keitz and Vogel, 1982

10.5.6.7 Supercritical Water, Advanced Electric and Vertical Tube Reactors

These incineration methods are basically in the developmental stage and even though they seem to be very promising further testing is required before these processes can be commercialized. The supercritical water process involves thermal destruction of organics in waste water based on the ability of many organic compounds to dissolve in super critical water. The process can probably be applied to dilute organic wastewaters (5 to 10 percent by weight) (Ross, 1984).

The vertical tube reactor is a very unique system for the destruction of organic wastes in a deep well with the appropriate temperature and pressure. The advanced electric reactor is also a unique design to treat organic substances such as PCBs and others. The process is based on a high temperature fluidized bed reactor (Ross, 1984).

10.5.7 Environmental Controls

10.5.7.1 Air Pollution Controls

Sources of air pollutants in hazardous waste incinerators include products of incomplete combustion of organic constituents and conversion of certain inorganic constituents resulting in gaseous or particulate contaminants.

Wet scrubbers are air pollution control devices that use a scrubbing fluid to wash contaminants from a gas. Both gaseous pollutants and particulates may be removed, although particulates may be more cost-effectively removed using other equipment in some cases (Peacy, 1984).

Electrostatic precipitation is a process by which particles suspended in a gas are electrically charged and separated from the gas stream on collecting plates. Both dry and wet electrostatic precipitators are available. Dry electrostatic precipitators have high efficiency for removal of particulates. The wet electrostatic precipitator can theoretically remove organic fumes as well as fine particulates (Peacy, 1984; Kiang and Metry, 1982).

After burners are basically simple combustion chambers used to burn gases being emitted from the incinerators.

Fabric filters or baghouses are air pollution control devices, consisting of a series of larger tubular bags which remove particulates from gases. Baghouse filters can be 99 percent effective in the removal of particulates, provided they are kept clean.

Gaseous pollutants can be removed from flue gases using one of three devices: spray towers, packed-bed towers or plate towers. All are mass transfer devices for gas absorption. Spray towers have lower removal efficiencies than that of packed and plate towers and are seldom used for gas removal (Kiang and Metry, 1982).

10.5.7.2 Heat Recovery

There are a variety of methods for recovering waste heat through various types of heat exchangers. The most commonly used device is the waste heat boiler. Each type of boiler has its own operating characteristics and can be used to develop steam or hot water.

Another device for energy recovery is the turbine generator. It is more costly than the heat exchangers but has more versatility in terms of product usage (Peacey, 1984).

10.5.7.3 Water Pollution Control

When water scrubbers are used in an incineration system, the acid scrubber water must be neutralized prior to discharge.

10.5.8 Overall Operation and Design Considerations

The overall design of a hazardous waste incinerator requires the evaluation of many factors including:

- Transportation and unloading
- Waste segregation
- Toxicity, flammability and explosiveness of wastes
- Storage
- Monitoring
- Emissions control
- Residue handling and disposal
- Other environmental factors.

10.5.9 Costs

It is very difficult to calculate the cost of incineration because of the high degree of complexity of the problem. The basic factors involved are:

- The limited industrial experience with incineration of bulk quantities of wastes

- The differences in type of waste, operation and design of an incinerator

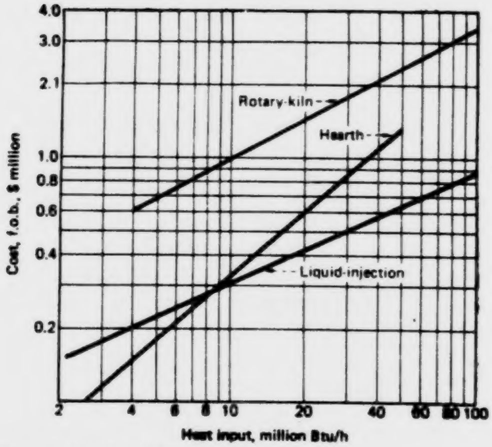
- The difficulty in estimating all capital, operating and maintenance costs.

The capital cost is comprised basically of the cost of purchased equipment and installation. The first is a firm cost and the second can vary depending on the geographic location, the assembly of control devices, topography and the availability of utilities. It consists of the costs of labor and material for foundations, structural supports, handling and erection, electrical insulation, construction, permitting and test burn protocol, etc. The purchased equipment costs are comprised of the costs of auxiliary equipment, instrumentation and control devices (Monsanto Research Corp., 1981). Annual operating and maintenance costs consist of the cost of labor; material, including fuel oil and chemicals; residual ash and waste water disposal; taxes; insurance; overhead; etc. These costs also include depreciation over the life of the facility and depend on the depreciation method and interest rate of the loan. Estimated annual operation and maintenance costs vary depending on size and characteristics of the waste stream, size and mechanical complexity of the incinerator and how it is used. Maintenance costs usually run about 5% of the depreciable capital cost.

10.5.9.1 Examples of Cost Estimates

Figure 10-55 depicts approximate capital costs for three basic types of incinerators as a function of thermal input. To obtain the real cost of the installed facility the numbers from the figure should be multiplied by 1.5. The costs of multiple hearth and liquid injection incinerators are similar for the heat input ranging from 5 to 10 MBtu. The rotary kiln is three times as

FIGURE 10-55. GENERAL ESTIMATES OF COSTS FOR THREE PREVALENT TYPES OF INCINERATORS



Source: Vogel and Martin, 1983

expensive for the same heat input (Vogel and Martin, 1983). The operating and maintenance costs add several hundred thousand dollars to the capital costs (Vogel and Martin, 1984).

The estimation of capital costs of an exemplary rotary kiln incinerator is given in Table 10-19. The annual operation and maintenance cost for the same incinerator are listed in Table 10-20.

Estimated capital and O&M costs for a liquid injection incinerator are shown in Table 10-21 and 10-22, respectively. Table 10-23 shows another estimate of O&M costs for a liquid injection incinerator based on the raw material requirements shown in Table 10-24. These costs were derived using a cost estimation model (McCormick, 1983) and are considerably higher than costs shown in Table 10-22.

TABLE 10-19. ESTIMATED CAPITAL COST FOR A ROTARY KILN

Item	1985 Cost $*
Combustion chambers:	
Refractory	$ 214,840
Shell	207,575
Burners	83,030
Water-storage system (two 10,000-gal tanks)	122,470
Waste-feed system (screw feeder)	10,379
Ash-handling system	62,272
Energy-recovery boiler	200,984
Air-pollution control system (quench chamber, scrubber and absorber)	425,530
Blower, 304 stainless steel	126.620
Stack (carbon steel, 90 ft at $90/ft)	8,406
Breeching (refractory-lined, 30 ft. at $300/ft)	9,341
Total equipment cost	$1,571,447
Installation (50% of total equipment cost)	786,254
Startup (10% of total equipment cost)	157,238
Spare parts (8% of total equipment cost)	125,790
Engineering (7% of total equipment cost)	110,118
Instrumentation (20% of total equipment cost)	314,580
Total capital cost	$3,065,467

Adapted from Vogel and Martin, 1984.

*Costs updated from 1983 to 1985 dollars using ENR Construction Index.

TABLE 10-20. ESTIMATION OF ANNUAL OPERATION AND MAINTENANCE COST FOR A ROTARY KILN

Item	1985 Cost $/yr*
Personnel (1 supervisor, 2 operators, 4 yard-crew workers, 1 secretary)	$ 151,322
Electricity (473 hp) (7,200 h/yr)	130,772
Water (3.1×10^8 gal/yr)	257,393
Auxiliary fuel	
Startup (10) (8 h) (10×10^6 Btu/h)	4,670
Operating (4.8×10^6 Btu/h) (7,200 h/yr)	430,407
Chemicals (2.25×10^6 lb lime/yr)	39,128
Effluent disposal:	
Scrubber liquid (3.1×10^5 gal/yr)	386,089
Ash (2.88×10^6 lb/yr)	14,945
Laboratory	62,272
Maintenance (10% of total equipment cost)	157,238
Refractory replacement (8-yr life)	26,984
Direct operating cost	1,661,220
Value of recovered steam	1,245,450
Net operating cost	415,770

Adapted from Vogel and Martin, 1984.

*Costs updated from 1983 to 1985 dollars using ENR Construction Index.

TABLE 10-21. CONCEPTUAL LEVEL CAPITAL COST ESTIMATES FOR A LIQUID INJECTION SYSTEM

Components	Cost
Site Acquisition and Development	$ 500,000
Tanks	150,000
Pumps, Piping, and Compressors	35,000
REceiving Station and Compressor Enclosure	70,000
Control Room, Auxiliaries, and Laboratory	150,000
Liquid Injection Incinerator	175,000
Scrubbing System	175,000
Total Installed Cost:	1,255,000
Construction, Overhead, and Fee	125,000
Contingency	125,000
Permitting	100,000
Start-up and Spare Parts Inventory	25,000
Total Project Cost:	$1,630,000

TABLE 10-22. CONCEPTUAL LEVEL OPERATIONS AND MAINTENANCE COST ESTIMATES FOR A LIQUID INJECTION SYSTEM

Components	Cost
Labor and Supervision (2 shifts/day)	$100,000
Fuel, Electric Power, Water and Chemicals and Caustic Soda)	75,000
Ash and Wastewater Disposal (assuming use of existing wastewater treatment plant)	5,000
Waste Analysis	30,000
Insurance, Taxes, and Overhead	50,000
Maintenance[1]	40,000
Depreciation[2]	75,000
Total Annual O&M Cost:	$375,000

[1] Maintenance costs are difficult to predict due to numerous and complex factors. Conceptual level estimates for a liquid injection system are typically five percent of depreciable capital costs, or, as in this case, approximately $40,000/yr.

[2] Annualized capital costs depend on how the system is depreciated and the interest rate if a loan is taken out instead of taking from cash flow. Assuming a 10 year straight-line depreciation, annualized capital cost estimates are approximately $75,000/yr. Given the current tax codes, such costs must take into account investment tax and energy recovery credits, as well as other corporate income tax adjustments.

Source: Star, 1985

TABLE 10-23. ESTIMATED ANNUAL O&M COSTS AND CREDITS FOR A LIQUID INJECTION INCINERATOR

Item	1985 Annual Cost
Natural gas	$ 36,326
No. 2 fuel oil	1,245
Power	72,665
Water	12,450
Caustic soda solution (50 wt %)	118,490
Liquid nitrogen	5,397
Sewer	60,197
Labor	166,060
Maintenance	77,841
Depreciation	155,681
Insurance/taxes	62,272
Total	$768,624

Source: McCormick, 1983.

Costs updated to $1985 using 1983 and 1985 ENR Construction Cost Indices.

TABLE 10-24. ESTIMATED RAW MATERIAL/UTILITY REQUIREMENTS

Item	Normal Rate	Total Annual Quantity
Fuel		
Natural gas for flame stabilization	1000 scfh	7 $\bar{M}$ ft^3
No. 2 fuel oil for startup	100 gal/startup	1400 gal
Power		
Id fan	95 hp	–
Compressor	70 hp	–
Blower	35 hp	–
Pumps	20 hp	–
Agitators	nil	–
Total	220 hp	1.15 Gwh
Water	110 gpm	48 $\bar{M}$ gal
Caustic soda solution (50 wt %)	230 lb/hr	1.6 $\bar{M}$ lb
Liquid nitrogen	38 ft^3/hr	270 M ft^3
Sewer use	110 gpm	45 $\bar{M}$ gal

Source: McCormick, 1983

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SECTION 11

CONTAMINATED WATER SUPPLIES AND WATER AND SEWER LINES

Hazardous substances can enter public water systems through a wide variety of pathways, contaminating the components of the systems as well as the water. Once contaminated, water systems can serve as secondary sources of contamination, and the systems' users can be exposed to hazardous substances over long periods of time.

Sanitary and storm sewers can become contaminated by infiltration of leachate or contaminated groundwater through cracks, ruptures, or poorly sealed joints in piping and by direct discharges into the lines. Potable water supply mains can become contaminated by contact with contaminated water that may inadvertantly flow through them, or by infiltration of leachate or contaminated groundwater. However, water mains are less susceptible to the infiltration of contaminants, since they are generally full-flowing, pressurized systems. The public health consequences of the contamination of municipal mains carrying potable water supplies to commercial and residential consumers are potentially much greater than the consequences of the contaminated sewage flowing to a treatment plant or of surface runoff draining to surface waters.

This Section presents methods for providing water supplies of acceptable quality with the minimum disruption of service; the methods are as follows:

- Water supply replacement:
  - New central water supply
  - Point-of-use water supplies
- Water Treatment:
  - Central water treatment
  - Point-of-use water treatment
- Alteration of water and sewer pipelines:
  - Replacement
  - Inspection and leak detection
  - Cleaning
  - Repairing and lining.

11.1 Replacement of Contaminated Central Water Supplies

Replacement of central water supplies, or sources of water that serve many users through central distribution systems, generally involves one or more of the following approaches:

- Purchase of water from another supply
- Provision of a new surface water intake(s)
- Provision of a new groundwater well(s).

The contaminated water supply may be abandoned or may be blended with the new supply to achieve acceptable water quality by dilution. Combinations of the approaches may be employed either concurrently (multiple replacement supplies) or consecutively (emergency water purchased from a neighboring supply unit, followed by new wells or intakes.)

Purchase of treated water from another supply requires a cross-connection(s) between the systems. Many neighboring public water departments, authorities, and companies maintain networks of interconnections that allow ready flow and meeting between systems for emergencies such as droughts, fires, line breakage, or malfunction of treatment facilities. Where cross-connections do not exist, water transmission lines can be installed. The information provided in Section 11.5 generally applies to new water pipelines. Numerous references are available that guide the design and installation of water transmission and distribution systems, including Fair (1971) and American Society of Civil Engineers (ASCE, 1975).

Provision of new surface water intake may be feasible where a groundwater source is to be replaced or where a replacement surface water intake would hydraulically isolate the water supply system from contaminated surface water (e.g., intake upstream of the source of contamination).

Surface water is drawn from rivers, lakes, and reservoirs through relatively simple submerged intake pipes, or through fairly elaborate towerlike structures that rise above the water surface. Important in the design and operation of intakes is that the water they draw be as clean, palatable, and safe as the source of supply can provide. River intakes are constructed well upstream from points of discharge of sewage and industrial wastes. Optional location should take advantage of deep water, a stable bottom and favorable water quality, all with proper reference to protection against floods, debris, ice, and river traffic. Small streams may be dammed up by diversion or intake dams to keep intake pipes submerged and preclude hydraulically wasteful air entrainment. Lake intakes are sited with due reference to sources of pollution, prevailing winds, surface and subsurface currents, and shipping lanes. Shifting the depth of draft makes it possible to collect clean bottom water when the wind is offshore, and, conversely, clean surface water when the wind is onshore. Reservoir intakes resemble lake intakes but generally lie closer to shore in the deepest part of the reservoir. They are often incorporated into the impounding structure itself (Fair, 1971).

The feasibility of providing new surface water intakes is dependent on numerous case-specific requirements and conditions, summarized as follows:

- Proximity of the point of intake to the water supply system
- Peak demand flow versus historic and predicted low flow in the water body
- Downstream environmental, recreational, and commercial effects of reduced flow
- Quality of the surface water and corresponding treatment requirements.

Provision of new groundwater wells is often feasible where the extent of aquifer contamination is relatively confined and would not be expected to be drawn to the area of influence of the new wells, or where other (usually deeper) aquifers can be tapped as a replacement water supply. The information provided in Section 5.1 generally applies to the design and construction of new groundwater wells.

11.2 Point-of-Use Water Supplies

Central water supplies that are contaminated at the source or in transmission through pipelines can be replaced permanently or temporarily with an independent supply at each point of usage. Such supplies could include one or a combination of the following:

- Bottled and bulk water
- Point-of-use wells
- Collection of rain water.

The use of bottled and bulk water is common for temporary or semi-temporary water supplies on an emergency basis until more permanent water supply arrangements can be made. Bottled water is widely available in small quantities from common retail outlets (grocery and drug stores) and in large quantities from commercial distributors. Larger bottles (e.g., five-gallon "water cooler" bottles) require dispensers in order to be conveniently used. Their full weight (approximately 50 pounds) may present handling and change-over problems for some users.

Bulk water can be provided in portable tanks (trailers or tank trucks) by commercial, clean water contractors and by public emergency service organizations (e.g., Army National Guard). Tanks normally used for other purposes, such as milk tank trucks, have also been used. The tanks are typically made available to homeowners at temporary, centrally located distribution points, where small containers can be filled for home use. Whole tanks can be made available to commercial and institutional establishments.

Point-of-use wells, or individual wells for each user establishment, may be feasible as a permanent alternative to a contaminated central supply, provided that the available groundwater is and can be expected to remain non-contaminated. The information provided in Section 5.1 generally applies to development of new groundwater wells.

Rainwater is rarely the immediate source of municipal water supplies, but could serve as a replacement to a contaminated water supply. The use of rainwater is generally confined to farms and towns in semiarid regions devoid of satisfactory groundwater or surface water supplies. For individual users, rainwater running off the roof is led through gutters and downspouts to a cistern situated on or below the ground. Cistern storage converts the intermittent rainfall into a continuous supply. For municipal service, roof water may be combined with water collected from sheds or catches on the surface of ground that is naturally impervious or rendered so by grouting, cementing, paving, or similar means (Fair, 1971).

The gross yield of rainwater supplies is proportional to the receiving or drainage area and the amount of precipitation. Because of the relatively small catchment area available, roof drainage cannot be expected to yield an abundant supply of water, and a close analysis of storm rainfalls and seasonal variations in precipitation must be made if catchment areas, standby tanks, filters, and cisterns are to be proportioned and developed properly (Fair, 1971).

11.3 Treatment of Contaminated Central Water Supplies

Central water supplies that are contaminated at the source can be treated to acceptable quality at central treatment systems. For some supplies, such as in small communities that pump groundwater directly to distribution systems without treatment, central treatment may require installation of new facilities. For other supplies, such as in large communities that already treat surface water before distribution, upgrading of existing treatment with the installation of polishing units may be necessary (Morrison, 1981).

Available water treatment methods include physical, chemical, and biological technologies, and combinations of these methods may be used for removal of some contaminants.

Many of the technologies described in Section 10-1, for treatment of aqueous wastes also apply to treatment of contaminated water supplies. In general, however, those technologies that are normally associated with "polishing" (i.e., removal of low concentrations of contaminants), such as activated carbon, ion exchange, and reverse osmosis, are most applicable to treatment for public water supplies.

11.4 Point-of-Use Water Treatment

11.4.1 General Description

Central water supplies that are contaminated at the source or in transmission can be treated to acceptable quality at the point of use (POU) with a variety of commercially available systems. Most applications of POU treatment units are for aesthetic purposes (taste, odor, and color), although their use is increasing for removal of organic contaminants from drinking water (Anderson, 1984a).

POU units are generally used in one of the following situations in residential applications:

- Line-bypass, where separate faucets are provided for treated and non-treated water; treated water is generally used for drinking, cooking, etc.
- Faucet-mounted, where all water passing through the faucet is treated.
- Whole-house, where all water entering the house is treated.

Line-bypass systems afford a compromise, providing only for treatment of water to be consumed, thereby minimizing treatment demands and costs.

POU treatment processes include the following (Anderson, 1984a; Perry, 1981):

- Activated carbon
- Activated alumina
- Reverse osmosis
- Ion exchange
- Distillation
- Ozonation
- Ultraviolet irradiation.

Of these processes, activated carbon is the most widely used and accepted process. Reverse osmosis and ion exchange are also widely available for applications where more stringent water quality requirements apply (hospitals, laboratories, etc.). Section 10.1 should be consulted for the applications and limitations of these methods.

11.4.2 Applications/Limitations

POU units are designed to remove a wide variety of contaminants from water. Manufacturers' claims vary, and numerous studies have been conducted to test the effectiveness of various units (Anderson, 1984a and b; Perry, 1981; Taylor, 1978).

Table 11-1 summarizes the general applications and limitations of the commonly used types of POU units. A major limitation common to all POU units is their reliance on the user (or service contracted by the user). If the units are not properly installed, operated, and maintained, the desired treatment may not be affected, or the accumulated contaminants may be released from the treatment unit after the treatment material is exhausted. (Taylor, 1978).

TABLE 11-1.
APPLICATIONS AND LIMITATIONS OF COMMONLY USED POINT-OF-USE TREATMENT UNITS

Process	Applications	Limitations
Activated carbon	• Organics, hydrocarbons, chlorine, trihalomethanes (THM), some pesticides	• Potential for excess growth of bacteria (Taylor, 1978) • Short-lived effectiveness for some contaminants (chlorine, THM, pesticides) (Taylor, 1978) • Potential desorption (release of contaminants) following exhaustion of carbon (Taylor, 1978)
Reverse osmosis	• Fluoride, total dissolved solids, sodium, sulfate, salts, metals	• High-pressure required to affect filtration • Low flow rate capacity requires storage tank and/or multiple systems in parallel
Ion exchange	• Dissolved minerals, metals, most inorganics	

11.4.3 Design Considerations

The primary design considerations for POU units are 1) selection of the appropriate units for the contaminant(s) of concern, 2) selection of the appropriate hydraulic capacity for the application, and 3) provision of appropriate criteria and schedules for maintaining the units. This information is generally available from the manufacturer or supplier.

Manufacturers' and suppliers' performance information should be confirmed by laboratory or trial tests under any of the following circumstances:

- Information has not been confirmed by a reputable organization or laboratory (National Sanitation Foundation (NSF), 1981)
- Disparate contaminants are present
- Concentrations of contaminants vary widely over time
- Contaminants and concentrations pose a high risk to human health
- Large numbers of units are to be employed.

11.4.4 Construction/Implementation Considerations

Installation procedures are provided by the manufacturer of each POU unit. Installation should be made by a licensed plumber and/or approval of the installation should be given by a local plumbing inspector. This is particularly important for whole-house units and by-pass units to ensure that backflow and inappropriate cross-connections are averted.

11.4.5 Operation, Maintenance, and Monitoring

Once installed, POU units operate relatively passively and require little or no attention. Proper maintenance and monitoring, however, are essential to the effectiveness and safety of the units. Maintenance generally consists of changing the cartridges on a regular schedule. However, few units give any detectable indication of having reached capacity. Conservative change-over schedules are recommended to help ensure that the units continuously serve their intended purpose. Alternatively, frequent monitoring of the quality of the treated water could be conducted by sampling and analysis to identify the need for cartridge changes. Many full-service water treatment companies provide installation and maintenance services.

11.4.6 Technology Selection/Evaluation

The selection of the appropriate POU unit is based largely on the contaminants to be removed. The applicability of the commonly used type of POU units is provided in Table 11-1. Reliability of performance should be a major consideration in evaluating POU units relative to other water supply or treatment technologies, as their reliance on the user or user-contracted services for installation and maintenance does not necessarily ensure a continuously safe water supply.

11.4.7 Costs

Typical initial equipment, installation, and monthly maintenance costs for POU treatment devices are shown in Table 11-2. Maintenance costs include changing of treatment cartridges on schedules consistent with the units' capacities and residential rates of water consumption.

TABLE 11-2.
1985 COSTS FOR POINT-OF-USE WATER TREATMENT SYSTEMS

Type of System	Initial Costs	Maintenance Cost
Activated carbon	$300-400/unit	$2-3/month
Activated alumina	$200-400/unit	$1-4/month
Reverse osmosis	$550/unit	$7-11/month
Deionization	$700/unit	$4-6/month
Combined activated carbon	$700-800/unit	$8-12/month

Source: Anderson, 1984b; Consumers Union 1984; Ingram, R., Culligan Water Conditioning of Greater Washington, Vienna, VA, personal communication, March 1985.

11.5 Replacement of Water and Sewer Lines

11.5.1 General Description

Replacement of water and sewer pipelines that are contaminated by contact with hazardous substances is seldom more cost-effective than rehabilitation, but may often be the only practical alternative. Replacement involves excavation of trenches, laying of new lines with noncontaminated pipe materials, laying new connections and/or tying in connections, and associated backfilling and surface restoration. Contaminated pipelines may either be abandoned in-place or removed in the course of trench excavation. Construction of water and sewer lines is common in land development projects and the associated methods, materials, and equipment are well-established.

11.5.2 Applications/Limitations

Pipeline replacement is applicable to virtually all cases of pipeline contamination. Excavation and replacement of defective sewer pipe segments is normally undertaken when the structural integrity of the pipe has deteriorated severely; for example, when pieces of pipe are missing, pipe is crushed or collapsed, or the pipe has large cracks--especially longitudinal cracks, and alternative rehabilitative techniques are not feasible. In addition, pipeline replacement is often required when the pipe is significantly misaligned (Water Pollution Control Federation (WPCF), 1983). Factors that would limit the feasibility of pipeline replacement are:

- Disruption of service and interim provisions
- Accessibility of pipeline and connections
- Interference of other utilities
- Disruption of vehicular traffic
- Depth of excavation
- Soil and groundwater
- Costs.

The primary disadvantage of pipeline replacement is the high cost. Analyses to determine the cost-effectiveness of pipe replacement must include all costs associated with the replacement. These costs typically include pavement removal and replacement; excavation; possible substitution of select backfill to replace poor quality existing material; dewatering and shoring, pipe materials and couplings, and traffic control. Potential cost increases resulting from interference with other underground utilities and narrow casements or limited space for construction must also be considered. In addition, consideration must be given to the need for temporary flow rerouting to maintain service. Depending on the service life assumed for other rehabilitation methods, the possible higher capital costs may be somewhat offset by the longer service life a new line provides (WPCF, 1983).

11.5.3 Design Considerations

In general, new pipeline systems will mimic the systems that they replace (size, material, grade, location, capacity, etc.). The considerations that govern the design of new systems will apply, but not control. Logistics and the presence of fixed constraints will dictate how the replacement system is designed. However, the need for replacement may provide an "opportunity" to upgrade the systems in terms of capacity, improved materials and methods, location, and/or direction of flow, and some consideration would be given to criteria for the design of new systems (WPCF, 1983).

The design of water distribution and sanitary sewer systems is addressed in numerous manuals and guidebooks, including ASCE (1975 and 1976). Information that is needed as input to the design includes:

- Population drawing from or contributing to the system
- Per capita water demand or sewage discharge
- Commercial, industrial, and institutional demand or discharge
- Minimum and peak daily demand or discharge
- Fire-fighting requirements
- Soil, groundwater, near-surface and geologic conditions
- Topography and grades
- Locations of potentially interfering features (utilities, buildings, etc.).

ASCE (1976) recommends that estimates of sewage flow be based on consideration of the following:

- The design period during which the predicted maximum flow will not be exceeded, usually 25 to 50 years in the future.
- Domestic sewage contributions based on future population and future per-capita water consumption. If a more satisfactory parameter than water consumption is available, that parameter should be used.
- In some instances, maximum flow rates may be determined almost entirely by extraneous flows, the source of which may be foundation, basement, roof, or areaway drains, storm runoff entering through manhole covers, or infiltration.

- Commercial area contributions are sometimes assumed to be adequately provided for in the peak allowance for per capita sewage flows in small communities

- Industrial waste flows should include the estimated employee contribution, estimated or gauged allowances per acre for industry as a whole, and estimated or actual flow rates from plants with process wastes that may be permitted to enter the sanitary sewer

- Institutional wastes are usually domestic in nature although some industrial wastes may be generated by manufacturing at prisons, schools, hospitals, etc.

- Air-conditioning and industrial cooling waters, if permitted to enter sewers, may amount to 1.5 to 2.0 gpm per ton of nonwater-conserving cooling units

- Infiltration may occur through defective pipe, pipe joints, and structures. Design allowances should be larger (under some circumstances, very much larger) than those stipulated in construction specifications for which acceptance tests are made very soon after construction.

The relative emphasis given to each of the foregoing factors varies among engineers. Some have set up single values of peak design flow rates for the various contributory items listed above. It is recommended, however, that maximum and minimum peak flows used for design purposes be developed step by step, giving appropriate consideration to each factor which may influence design. (ASCE, 1976).

If a sewer is to transport stormwater or wastewater from one location to another, it must be constructed sufficiently deep (below the ground surface) to receive these flows from basic or service connections. It should be resistant to both corrosion and erosion and its structural strength must be sufficient to carry backfill, impact, and live loads satisfactorily. The size and slope, or gradient, of a sewer must be adequate for the flow to be carried and be sufficient to avoid deposition of solids. The type of sewer joint must be selected to meet the conditions of use as well as those of the ground. Economy of maintenance, safety to personnel and the public, and public convenience during its life and during construction also must be considered (ASCE, 1976).

The pipe material used for sanitary and storm sewers can influence other design decisions and should, therefore, be selected early in the design process.

Factors that should be considered in the selection of materials for both water and sewer construction are (ASCE, 1976):

- Flow characteristics-friction coefficient
- Life expectancy and use experience

- Resistance to scour
- Resistance to acids, alkalis, gases, solvents, etc. (sewers only)
- Ease of handling and installation
- Physical strength
- Type of joint - watertightness and ease of assembly
- Availability and ease of installation of fittings and connections
- Availability in sizes required
- Cost of materials, handling, and installation.

No single material will meet all conditions that may be encountered in sewer design. Selections should be made for the particular application and different materials may be selected for parts of a single project (ASCE, 1976).

New materials are continuously being offered for use in sewer construction. Some of the more commonly used materials are (ASCE, 1976):

- Asbestos cement
- Brick masonry
- Vitrified clay
- Concrete
  - Precast
  - Reinforced precast
  - Cast-in-place
- Iron
  - Cast iron
  - Ductile iron
- Fabricated steel
  - Corrugated
  - Plain
- Organic synthetic materials
  - Solid-wall plastic (polyvinyl chloride (PVC), polyethylene, acrylonitrile-butadiene-styrene (ABS), fiberglass reinforced plastic)
  - Truss pipe
  - Corrugated polyethylene.

Estimates of flow and system pressure in water distribution systems are based on the following (Fair, 1971):

- Domestic, industrial, and other normal uses, determined in a manner similar to estimating sewage flow
- Standby requirements for fire-fighting as required by local codes and/or the American Insurance Association
- Estimates of system leaking
- Calculation of in-transit frictional pressure losses.

A variety of pipe materials are also available for water systems; among the most common are (ASCE, 1975):

- Iron
  - Ductile iron
  - Cast iron
- Concrete
- Asbestos - cement
- Steel
- Organic synthetic materials:
  - Polyvinyl chloride (PVC)
  - Polyethylene (PE)
  - Acrylonitrile - butadiene - styrene (ABS).

11.5.4 Construction/Implementation Considerations

A variety of conventional and nonconventional methods are available for constructing water and sewer lines. The most common method is open-trench excavation, which often requires lateral bracing of trench walls in deep cuts and/or non-cohesive soils. This method of sewer construction is described in ASCE, (1976). Other methods of construction include:

- Augering, or boring, where the pipe is pushed through the soil and the soil ahead of the pipe is removed by an auger that is advanced with the pipe
- Jacking, where the pipe is pushed through the soil and the soil ahead of the pipe is removed by laborers working from inside the pipe
- Tunneling by various means.

11.5.5 Operation, Maintenance, and Monitoring

Replacement water and sewer lines require no operations, maintenance, or monitoring beyond that required for other newly constructed lines. For water lines, this includes flushing, leak detection and repair, and hydrant testing to ensure that fire-fighting requirements are met (Fair, 1971). For sewer lines, this includes occasional flushing and cleaning (see Section 11.6), removal of blockages, ensuring adequate ventilation, and leak detection and repair.

11.5.6 Technology Selection/Evaluation

Factors that generally favor the selection of line replacement over other technologies are the complete removal of contaminants from the system and the provision of new systems with associated functional lines. Major drawbacks of replacement are disruption of surface activities and costs. Performance and reliability of replacement systems are the maximum obtainable; i.e., new systems are the basis of comparison for evaluating other pipeline alteration technologies.

11.5.7 Costs

Typical costs for replacement of water and sewer lines ae provided in Table 11-3.

11.6 Inspection and Cleaning of Water and Sewer Lines

11.6.1 General Description

Available techniques for inspecting and cleaning sewer lines are generally applicable to water lines. However, the water lines are normally smaller in diameter than sewer lines, and size is often a limiting factor in the applicability of inspection and cleaning technologies. Inspection techniques include smoke testing, dye-water flooding, first-hand visual observation, and closed-circuit television visual observation.

Inspection is generally conducted to identify one or more of the following conditions:

- Points of groundwater leakage
- Structural defects in need of repair

TABLE 11-3.
1985 COSTS FOR REPLACEMENT OF WATER AND SEWER LINES

Item	Unit	Cost Per Unit
Sewer pipe, material and installation, in-place:	Linear foot	
8-inch diameter		$6-10
18-inch diameter		$13-31
36-inch diameter		$33-120
Water pipe, material and installation, in-place:	Linear foot	
2-inch diameter		$3-7
4-inch diameter		$5-11
12-inch diameter		$16-27
Pipe bedding material	Cubic yard	$14-25
Trench excavation, backfill, and compaction:		
Water lines	Linear foot	$1-3
Sewer lines	Cubic yard	$6-10

Source: Godfrey, 1984

- Points of connection
- Areas in need of cleaning.

Inspection of pipelines for leaks or infiltration points may be part of a regular sewer or water line maintenance program. Methods to detect and locate pipeline breaches include the use of dyes and other tracer chemicals, patented audiophone leak detectors, smoke testing, and installation of pressure gages along a given length of pipe to monitor changes in hydraulic gradient (Linsley and Franzini, 1979). The interiors of small diameter sewers and large diameter water lines are commonly inspected by pulling skid-mounted miniaturized closed-circuit television cameras through the line. The entire inspection can be recorded on videotape for future reference.

Cleaning of water and sewer line removes deposits and debris from the pipelines and is conducted for one or more of the following reasons:

- Improve flow conditions and capacity
- Allow visual inspections
- Provide clean surfaces for placement of repair materials.

Available sewer-cleaning techniques include mechanical scouring, hydraulic scouring and flushing, bucket dredging, suction cleaning with pumps or vacuum, chemical absorption, or a combination of these methods. Access to sewer lines for interior cleaning and repair is most commonly afforded by manholes. Basin inlets and service connections provide additional points of access. Service and fire hydrant connections allow access to municipal water lines.

11.6.2 Applications/Limitations

Pipeline inspection is applicable to all visually observable cases of pipeline contamination or leakage of contaminated water. The methods are well-developed and accepted. Small-diameter pipelines (less than 6 inches) cannot be inspected by closed-circuit television, and pipelines less than 48 inches in diameter cannot readily be inspected first-hand by workmen. Television inspection offers the advantages of worker safety and a permanent videotape record of the inspection. It is common practice to clean pipelines before inspection to ensure visibility of defects and free access of workmen and/or equipment.

11.6.3 Design Considerations

Design of inspection and cleaning operations for water and sewer lines consists primarily of planning for the logistics of implementation. Sections of pipeline to be inspected and/or cleaned are selected based on evidence of the presence of contamination or contaminated seepage; sections may be added or deleted in process, depending on interim findings. Critical points of operation such as access manholes, base of operation, and material storage are selected. Methods of managing disruption of service (water or sewer) and surface activities such as traffic are also planned. Affected parties are notified in advance of the planned work.

11.6.4 Construction/Implementation Considerations

Smoke bombs or canisters are used to generate the smoke required for smoke testing of pipelines. The smoke should be nontoxic, odorless, and nonstaining. Air blowers are used to force the smoke into the pipes. Smoke coming out of the ground, catch basins, pipes, and other sources during the test is noted and recorded by observers and photographs are taken for permanent documentation of the results. Sand bags and/or plugs can be used to block the sewer sections to prevent the smoke from escaping through the manholes and adjacent sewer pipes (WPCF, 1983).

Dye-water flooding is used to simulate rainfall and thus identify points of runoff-related infiltration and sources. Equipment needed for dye-water testing is limited to that required to carry the water to the testing site and to block the sewer sections to be tested. A fire hose is all that is needed to deliver the water to the testing site. When the water source is not close by, water tankers are required to deliver the water. Sand bags or sewer pipe plugs are normally used to block the sewer sections. Fluorescent dyes are usually employed for high visibility (WPCF, 1983).

The following is a general procedure for dye testing for possible infiltration from a storm sewer to a sanitary sewer; similar procedures are used for evaluating other sources:

- Plug both ends of the storm drain section to be tested with sand bags or other materials. Block all the overflow and bypass points in the sewer section. Provide bypassing of flow, if necessary.
- Fill the storm drain section with water from fire hydrants or other nearby water sources. Add dye to the water.
- Monitor the downstream manhole of the sanitary sewer system for evidence of dyed water.
- Measure the flows in the manhole before and during the dye-water testing. As an alternative, the flows can be simultaneously measured at both the upstream and downstream manholes during the test.
- Record the location of storm drains and sanitary sewer lines being tested; the time and duration of tests; the manholes and the flow rates where the flows are monitored; the observed presence, concentrations and traveltime of the dyed water into the flow monitoring manholes; and the soil characteristics (WPCF, 1983).

First-hand visual observation of conditions is possible in large diameter sewers that permit workmen to enter. Physical proximity to the pipeline interior enables workman to observe structural conditions, condition of joints, location and nature of deposits and debris, and locations of points of infiltration. Worker safety is an important consideration under such conditions (WPCF, 1983).

Television inspection is accomplished by using closed circuit systems specifically designed for sewer inspection. There are several configurations of closed-circuit systems for sewer inspections, each of which have in common the following (WPCF, 1983):

- Power for operation generated on-site
- Power control
- Transport winches
- Video (color, if possible) and lighting control
- Recording and documentation
- Radio communication

Television inspection provides a video screen picture of pipeline being inspected to an operator in a nearby van. The operator controls the speed of movement along the pipeline and records details of conditions observed. Most systems also provide for videotape recording of the inspection for future reference.

Pipeline cleaning is necessary for efficient collection system operation and for exposing pipe materials for television inspection. All collected sediment and debris should be removed from the line and disposed of at an approved site. Care should be taken during the cleaning to minimize transport of deposits into downstream lines. As extensive outline of sewer cleaning methods can be found in WPCF (1982). The procedures for the most common methods are briefly described below.

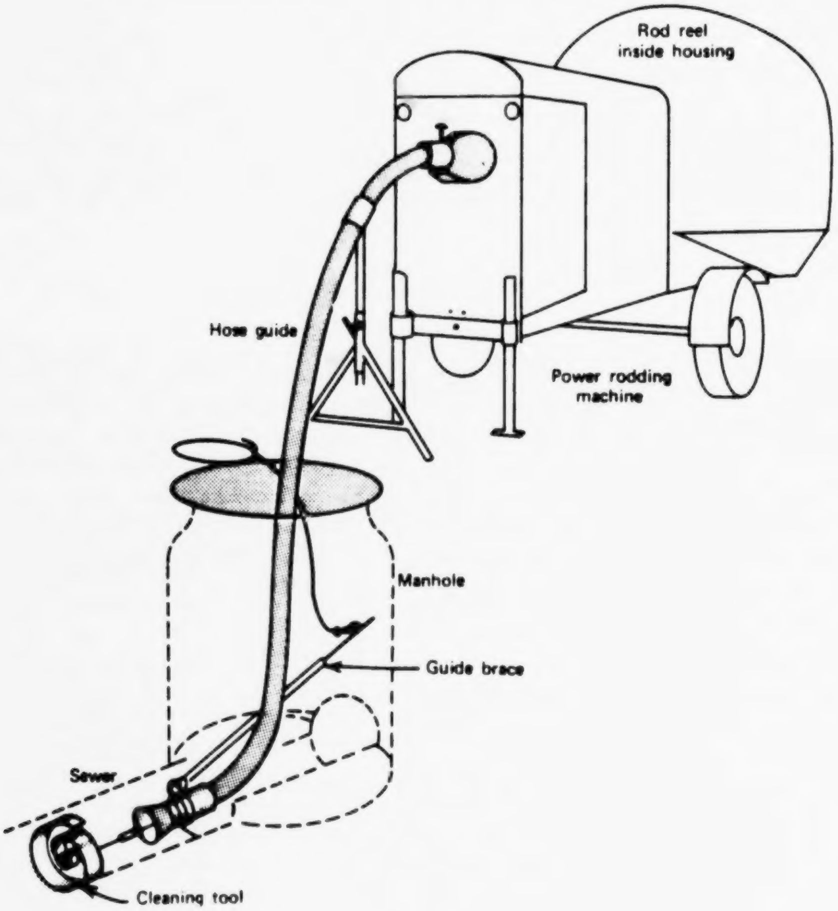
Mechanical scouring techniques include the use of power rodding machines ("snakes"), which pull or push scrapers, augers, and brushes through the obstructed line (Figure 11-1). "Pigs," bullet-shaped plastic balls lined with scouring strips, are hydraulically propelled at high velocity through water and sewer mains to scrape the interior pipe surface.

Hydraulic scouring is achieved by running high-pressure hoses into sewer lines through manholes and flushing out sections of the sewer. This technique is often used after mechanical scouring devices have cleared the line of solid debris or loosened sediments and sludges that coat the inner surface of the pipe.

A bucket machine can be used to dredge grit or contaminated soil from a sewer line (Figure 11-2). Power winches are set up over adjacent manholes with cable connections to both ends of a collection bucket. The bucket is then pulled through the sewer until loaded with debris. The same technique can be used to pull "sewer balls" or "porcupine scrapers" through obstructed pipes (Hammer, 1975). Bucket dredging is also useful for collecting samples of contaminated sediments, groundwater, or leachate that may have infiltrated the lines.

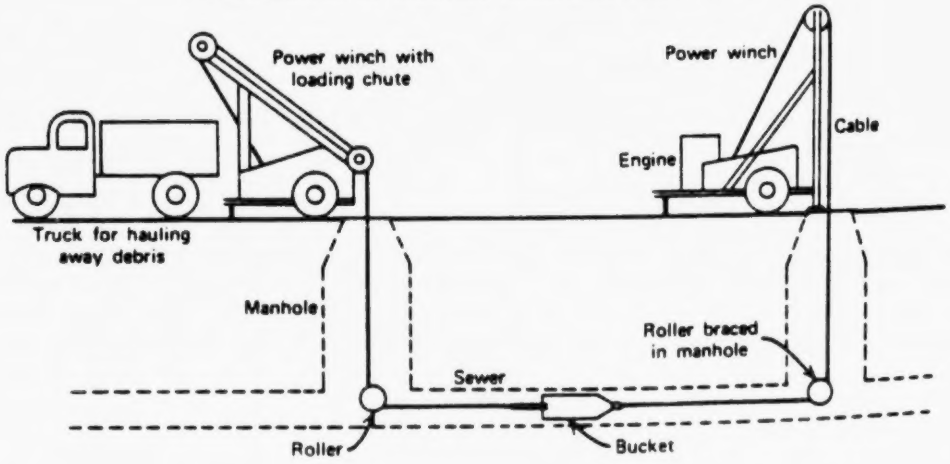
Suction devices such as pumps or vacuum trucks also may be used to clean sewer lines of liquids and debris. Again, manholes and fire hydrants provide easy access for the setup and operation of such equipment.

FIGURE 11-1.
POWER RODDING MACHINE



Source: Hammer, 1975

FIGURE 11-2.
SCHEMATIC OF BUCKET MACHINE CLEANING



Source: Hammer, 1975

Another method of sewer pipeline cleaning is the use of hydrophilic polymer foams and gels that absorb and physically bind liquid pollutants in a solid elastromeric matrix (Johnson, J., Chemical Research Division, 3m Company, personal communication, March 1980). These polymers are special chemical grouts that can either be applied internally to pipelines or injected through breaks in the line from the exterior. Once the absorbent grout has set (solidified), the solid grout/pollutant matrix can be hydraulically flushed from the line. The applications of any of these hydrophilic grouts, whose formulations are often proprietary, are still in the developmental and testing stages.

11.6.5 Operation, Maintenance, and Monitoring

Inspection and cleaning of water and sewer lines are essentially operating and maintenance activities. Montoring for effectiveness of cleaning may be warranted upon completion and periodically thereafter to ensure ongoing absence of contaminants from the pipeline.

11.6.6 Technology Selection/Evaluation

Inspection and cleaning of water and sewer lines is established and accepted technology for conventional applications. Removal of hazardous contaminants from pipelines may be afforded by conventional methods where contaminants are associated with deposits or debris such as sludge, slime, or sediments. Contaminants that are sorbed onto the pipe material may not be as readily removed. Information concerning the effectiveness of conventional methods in such cases is not available.

11.6.7 Costs

Inspection and cleaning of water lines can be accomplished by a variety of methods, and costs vary accordingly. Television inspection and light high-pressure water cleaning (the minimum required in preparation for repairing or lining pipelines) typically costs $100 to $150 per hour, or $0.40 to $0.60 per linear foot for a rate of progress of 2,000 feet per eight-hour day and $0.80 to $1.20 per linear foot at a rate of 1,000 feet per day. (D'Angelo, T., Pipe Maintenance Services, Inc., Exton, PA, personal communication, April 1985). Costs for other inspection and cleaning methods are highly variable and dependent on the type of pipeline and nature of the material being removed.

11.7 Rehabilitation of Water and Sewer Lines

11.7.1 General Description

Water and sewer lines that are in contact with contaminated substances or allow infiltration of contaminated water can be lined or sealed in-place with chemically inert material in order to isolate the water being transmitted from the contaminants. Available methods include the following:

- Insertion of a new pipe inside of existing pipe (sliplining)
- In-place forming of new pipe inside of existing pipe
- Point repairs of leaks and other defects.

Sliplining involves sliding a flexible liner pipe of slightly smaller diameter into an existing circular pipeline and then reconnecting the service connections to the new liner. Polyethylene is the most common material used for sliplining pipelines (WPCF, 1983).

A patented system called "inversion lining" uses a flexible lining material that is thermally hardened. Access to the pipeline can be made through manholes or excavations. After the lining system has been installed and cured, a special cutting device is used with a closed-circuit TV camera to reopen service connections. The system is available only through licensed contractors (WPCF, 1983).

Because inversion lining can be accomplished relatively quickly and without excavation, this method is particularly well-suited for repairing pipelines located under existing structures or large trees. It also is particularly useful for repairing pipelines located under busy streets or highways where traffic disruption must be minimized. Because the liner expands to fit the existing pipe geometry, this method is applicable to all pipeline shapes. The cured resin material is reportedly corrosion-resistant. Inversion lining also affords minor structural reinforcement. Inversion lining may be used for misaligned pipelines or in pipelines with bends where realignment or additional access is not required (WPCF, 1983).

Inversion lining using water to cure the resins is generally used in pipelines with diameters less than 57 inches and manhole-to-manhole segments less than 1000 feet long. Larger diameter pipelines (to 108 inches) have been lined by inversion techniques using air (WPCF, 1983).

Inversion lining is relatively new in the U.S. and its cost-competitiveness has not yet been fully established (WPCF, 1983). It is a patented system available only through a limited number of licensed contractors.

Chemical grouting is commonly used for sealing leaking joints in structurally sound sewer pipes. Small holes and radial cracks can also be sealed by chemical grouting (WPCF, 1983).

Chemical grouts are synthetic materials that are applied as low-viscosity liquids and cure as flexible, form-fitting solids. Commonly used chemical grouts are acrylamide gel, acrylate polymer, and polyurethane gel (WPCF, 1983).

11.7.2 Applications/Limitations

Repairing and lining of water and sewer lines applies to lines that are 1) contaminated as a result of ongoing contact with contaminated substances, or 2) allowing the infiltration of contaminated water. The application of materials to the interior of the pipe should resolve either or both problems if the materials are properly selected. The materials used have low susceptibility to chemical degradation and have relatively low permeability to water, and would be expected to effectively isolate sewage and water flows from contaminated pipelines and seepage. However, the repair and lining materials and techniques were not developed for control of hazardous contaminants and there is no information available that addresses their effectiveness under these special circumstances. Factors that could adversely affect the performance and reliability of repairs and lining are: 1) incompatability of repair materials and contaminants, and 2) permeability of repair materials with respect to contaminants.

Sliplining is used to rehabilitate extensively cracked pipelines, especially lines in unstable soil conditions. It is also used to rehabilitate pipe installed in a corrosive environment and in areas where sewer pipes have massive destructive root intrusion problems (WPCF, 1983). The flexible liner pipes have the advantage of being able to accomodate a normal amount of future settlement or, moderate horizontal or vertical deflection.

If the existing pipeline joints are offset, service lateral taps are protruding, or if the diameter of the pipeline has been significantly reduced in some other manner, the liner pipe diameter may need to be much smaller than the diameter of the existing pipeline. Such conditions can limit the utility of this method (WPCF, 1983).

Premanufacturered sliplining pipe cannot be used in pipe that is significantly "out of round," since its cross section must pass through that of the existing pipe. New pipe formed in-place cannot be used where the existing pipe has low structural integrity, unless reinforcing is added in the forming (generally limited to larger diameters where workmen can enter).

Point repairs can be used where problems are confined to joints or to relatively few short sections of pipe.

Chemical grouting is generally used to seal open pipeline joints and cracks. It does not improve the strength of a pipeline, and should not be used when pipe is severely cracked, crushed, or badly broken. Chemical grouts, once applied, may dehydrate and shrink if the surrounding moisture is reduced significantly. Some joints and cracks may be difficult to seal chemically using gel grouts when large voids exist outside the pipe joint and extremely large quantites of grout may be required to seal the joint (WPCF, 1983).

Inspection and cleaning of pipelines is generally necessary in preparation for rehabilitation. These methods are addressed in Section 11.6.

11.7.3 Design Considerations

Design of water and sewer line rehabilitation consists primarily of planning for the logistics of implementation. Sections of pipeline to be rehabilitated are identified based on television or other inspections. Critical points of operation such as access manholes, base of operation, and material storage are selected. Methods of managing disruption of service (water or sewer) and of surface activities such as traffic are also planned. Affected parties are notified in advance of the planned work.

11.7.4 Construction/Implementation Considerations

Typical sliplining materials include high-density polyethylene (HDPE) and fiberglass-reinforced pipe (FRP).

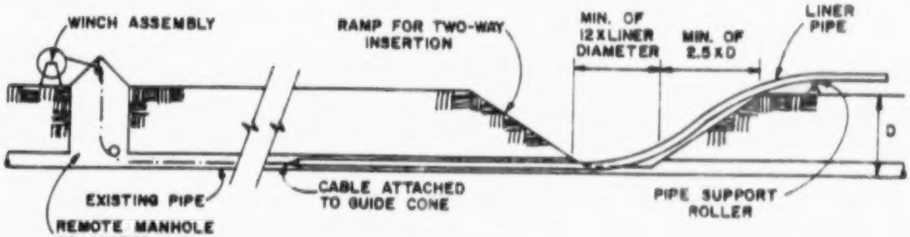
Before installing a liner pipe, the existing pipeline should be inspected by closed circuit TV to identify all obstructions such as displaced joints, crushed pipe, and protruding service laterals to locate service connections. The existing pipe is thoroughly cleaned immediately before sliplining begins (WPCF. 1983).

HDPE sliplining is pulled through existing pipelines by a cable that is fed through the section to be lined. The cable and pipe are advanced by a winch and pully assembly (see Figure 11-3). An approach trench is excavated at the insertion end of the existing pipe section to allow a gradual transition from the ground surface, where sections of HDPE pipe are heat fused to form a continuous pipe to an opened section of pipe. Several thousand feet of water or sewer line can be sliplined in a single set-up of such an operation (Waste Engineering and Management, 1983). FRP can be sliplined in a manner similar to that used for HDPE, although a combination of pushing and pulling of the pipe can be employed (see Figure 11-3) (A.O. Smith-Inland, undated).

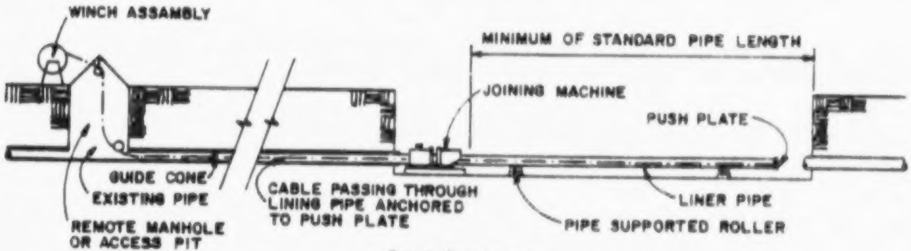
Wastewater flow in existing sewers may not need to be interrupted during insertion of a sliplining as low flows may continue through the annular space between the two pipes. Alternatively, it may be necessary to temporarily plug the upstream lines and pump the flow around the section being lined using above-ground piping (WPCF, 1983). The annulus between the old and new pipelines is sometimes filled with grout where structural failure of the old pipe could concentrate loads and cause problems with the HDPE pipelines.

In-place forming of new pipe inside of existing pipe is accomplished with portland cement grout and mortar, chemical grouts, and synthetic resins.

FIGURE 11-3.
SEWER SLIPLINING METHODS


WINCH ASSEMBLY
RAMP FOR TWO-WAY INSERTION
MIN. OF 12 X LINER DIAMETER
MIN. OF 2.5 X D
LINER PIPE
D
EXISTING PIPE
REMOTE MANHOLE OR ACCESS PIT
CABLE ATTACHED TO GUIDE CONE
PIPE SUPPORT ROLLER

"PULL" METHOD


WINCH ASSAMBLY
MINIMUM OF STANDARD PIPE LENGTH
JOINING MACHINE
PUSH PLATE
GUIDE CONE
EXISTING PIPE
REMOTE MANHOLE OR ACCESS PIT
CABLE PASSING THROUGH LINING PIPE ANCHORED TO PUSH PLATE
LINER PIPE
PIPE SUPPORTED ROLLER

"PUSH" METHOD

Source: WPCF, 1983

Chemical grouts can be used to seal fractures and leaking joints to waterproof points of infiltration/exfiltration. Grout materials used for this application include acrylamide, acrylate, urethane and polyurethane.

The chemicals necessary to form acrylamide or acrylate gels are usually mixed in tanks and pumped through separate hoses to the pipeline joint to be sealed. The water and catalyst solution initiates the chemical reaction when mixed with the acrylamide solution. Additives can be included in either solution to help control shrinkage, reaction or "gel" time, and other variables (WPCF, 1983).

The two solutions are pumped through separate hoses to the point to be sealed. The solutions are mixed as they are injected into the leaking opening, initiating the chemical reaction. This reaction changes the two solutions into a gel. The gel time can be controlled from just a few seconds to several minutes. The grouts or gels stabilize soil around joints or cracks by filling the voids (WPCF, 1983).

Urethane grout materials form either an elastomeric gel, much like the acrylamide and acrylate gels, or a rubber-like foam. Water is the catalyst for the urethane gel material. Urethane gel seals pipeline joints by forming a collar within the pipe joint as well as by consolidating soils and filling voids outside the joint (WPCF, 1983).

Urethane gel is applied in essentially the same manner as the acrylamide and acrylate gels.

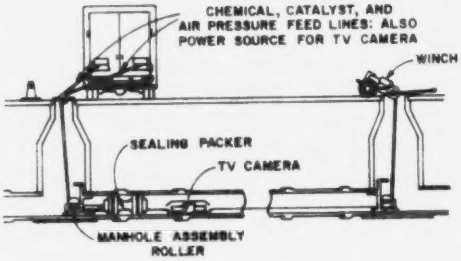
Polyurethane foam differs from the gel grouts in that the foam is used to form an in-place pipeline gasket and does not fill voids or stabilize the soil outside of the pipe joint (WPCF, 1983).

Small and medium diameter pipes can be grouted using a hollow metal cylinder with inflatable rubber sleeves on each end of a center band, called a "packer." An inflated packer can be used both to test and chemically seal a pipeline joint. A van is used as the operation and control center for a TV monitor, pumps, air compressors, and the feed system equipment. A closed-circuit TV camera allows positioning of the packer at pipeline joints and cracks for sealing. The packer and the TV camera are pulled along with a cable from manhole to manhole, and the process is viewed on the TV monitoring screen in the van (see Figure 11-4) (WPCF, 1983).

The amount of grout needed to seal a defect depends upon the size of the leak. The gels usually are pumped until the grout solidifies; the back pressure will then indicate to the operator that the leak has been sucessfully sealed. The rubber sleeves are deflated and moved to the next joint for sealing (WPCF, 1983).

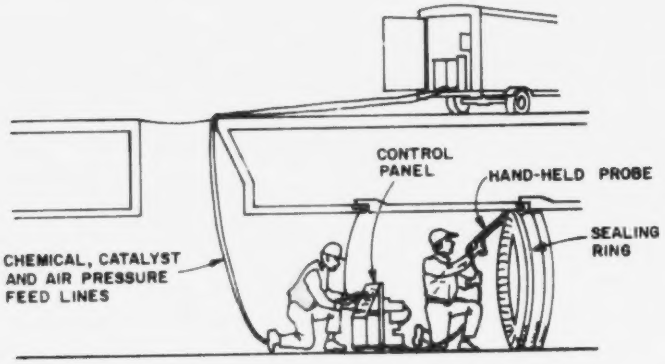
For grouting large-diameter pipes, pressure grouting or manual placement of oakum soaked with grout may be used. Pressure grouting is accomplished using pipe grouting rings or predrilled injection holes (Figure 11-5).

FIGURE 11-4.
TYPICAL ARRANGEMENT FOR APPLYING CHEMICAL GROUT TO SMALL DIAMETER PIPE


CHEMICAL, CATALYST, AND AIR PRESSURE FEED LINES: ALSO POWER SOURCE FOR TV CAMERA
WINCH
SEALING PACKER
TV CAMERA
MANHOLE ASSEMBLY ROLLER

Source: WPCF, 1983

FIGURE 11-5.
TYPICAL ARRANGEMENT FOR SEALING LARGE DIAMETER PIPE WITH GROUTING RINGS



Source: WPCF, 1983

Grouting using sealing rings requires the use of a small control panel, chemical and water pumps, and various other accessories depending on the type of sealing grout being used. A worker must enter the line, manually place the ring over the joint, and inflate the ring to isolate the joint. Sealing grout is pumped into the small void between the pipe wall and the face of the ring through a hand-held probe. As the pressure in the void increases, the grout solution is forced into the joint and surrounding soil. The catalyst solution is injected and the grout cures, sealing the joint from infiltration (WPCF, 1983).

Linings and coatings can be used to protect pipelines from internal corrosion. Most linings, however, are integrated into the pipe when it is made (WPCF, 1983).

Reinforced shotcrete (gunite) is a mixture of fine aggegate, cement, and water applied by air pressure using a cement ejector. Compared to cement mortar linings, gunite is denser and has a higher ultimate compressive strength. It also improves a pipeline's structural integrity. Gunite adheres well to other concrete and brick sewers and is more corrosion-resistant than normal concrete. It can be trowled to a finish to improve a pipeline's hydraulic characteristics (WPCF, 1983).

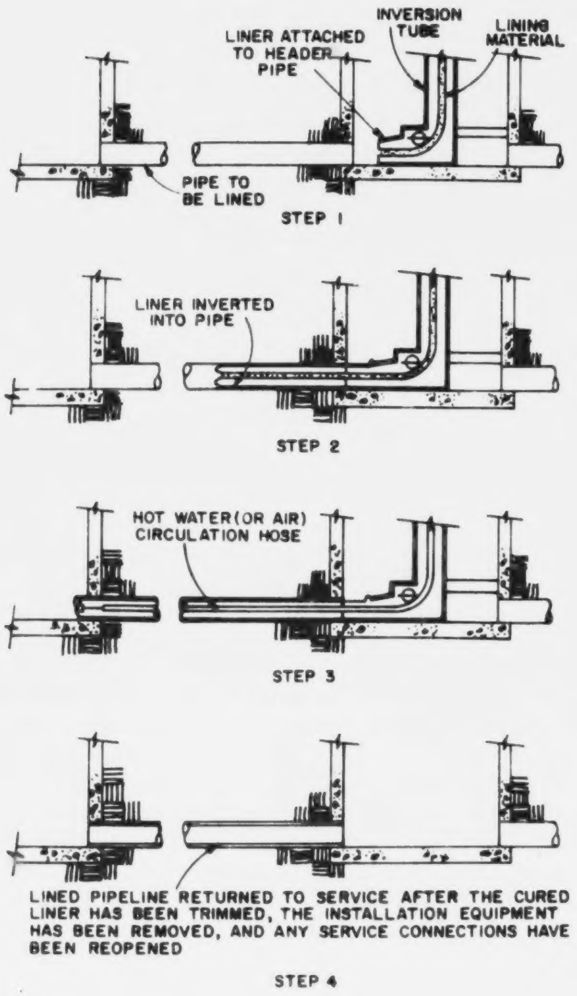
Gunite is well-suited for extremely deteriorated large sewers where persons and equipment can work without restriction. Long lengths of sewers may be effectively renewed with little excavation and minimal traffic disruption.

Gunite can be applied under low wastewater flows; however, totally dewatering the pipeline is more effective. Welded wire mat or small diameter rod reinforcing is used for structural gunite applications (WPCF, 1983).

In-place forming with synthetic resins (inversion lining) can be accomplished without excavation in most cases. The reconstruction is done through existing pipe access points, requiring only limited disruption of surface conditions and activities. A four-step installation process, shown in Figure 11-6, can normally be accomplished in a matter of days (Utz, 1983). A fiberfelt tube impregnated with a liquid resin is fed into an inversion standpipe which has been erected on site. The felt tube has an impermeable coating on the outside which eases handling and provides a water barrier for the inversion process (Insituform, undated). The end of the tube is pulled through the inversion standpipe, turned inside out and clamped to the standpipe such that a leak-proof seal is established. As more water is added to maintain the weight of the column, additional tubing is fed into the standpipe, and the impregnated tube snakes its way forward through the pipe being rehabilitated (WPCF, 1983).

The weight of the water presses the coated felt at the nose, inverts it, and then presses the resin-impregnated side against the insides of the existing pipe, leaving the smooth coated side as the new interior surface of the rehabilitated pipe. After the inverted tube reaches the next manhole or other access point, the water is heated to cure the resin, forming an impermeable new pipe within the old pipe. The ends are cut off, the head of

FIGURE 11-6.
INVERSION LINING INSTALLATION PROCEDURE



LINED PIPELINE RETURNED TO SERVICE AFTER THE CURED LINER HAS BEEN TRIMMED, THE INSTALLATION EQUIPMENT HAS BEEN REMOVED, AND ANY SERVICE CONNECTIONS HAVE BEEN REOPENED

Source: WPCF, 1983

water is released, and the operation is complete. Service connections are reinstated in non-man-entry pipes by means of remotely controlled cutting device (Insituform, undated).

11.7.5 Operation, Maintenance, and Monitoring

If properly implemented, lined, and repaired, water and sewer lines should require no operation or maintenance beyond that required for other lines. Periodic monitoring for contaminants of concerns in water or sewage should be conducted to determine whether leaking or other failure of the measures has occurred.

11.7.6 Technology Selection/Evaluation

Repairing and lining offer the advantage of eliminating contaminants without the need for disruption of surface activities. However, the presence of contaminants in close proximity to water or sewage may be cause for ongoing concern. Because these technologies were not specifically developed for control of hazardous substances, their performance and reliability under such special circumstances is neither certain nor can be fully evaluated with available information. Coordination of material selection with manufacturers may be useful in determining material compatability. Also, laboratory bench-scale and/or field pilot-scale tests may be warranted to ensure that effective, long-term isolation of contaminants can be affected.

11.7.7 Costs

Costs of sliplining water and sewer lines vary with the diameter and depth of the pipeline. Costs for relatively small diameter (less than 15-inch) HDPE sliplining projects range from $20 to $30 per linear foot (D'Angelo, T., Pipe Maintenance Services, Inc, Exton, PA, personal communication, April 1985; Metcalf, K. Norfolk, VA, personal communication, April 1985). Larger diameter sliplining projects are seldom undertaken and must be costed on a project-specific basis.

Inversion lining costs are normally given on a per-linear-foot basis for initial television inspection, cleaning, by-pass pumping, and post-construction television inspection combined. The following are representative unit costs for typical inversion lining of sewer lines.

Diameter (inches)	Cost (Linear Foot)
8	$45-50
10	$47-52
12	$49-54

Reconnection of laterals typically costs $100 to $250 each, depending on logistics and the number of laterals in a given project. (Tice, M., Insituform East, Inc. Landover, MD, Personal communication, April 1985.)

Grout repairs to sewer pipelines are generally accomplished by pumping grout into a joint until soil voids are filled, as determined by backpressure. A wide range of grout volumes can be pumped into a joint, and sewer grouting work is typically conducted on a per-hour basis for manpower and equipment ($100 to $150 per hour) and on a per-gallon basis for grout ($5 to $10 per gallon for chemical grout).

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APPENDIX B

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